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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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PLATES

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THE ST. LAWRENCE WATERWAY TO THE SEA*

BY FRANCIS C. SHENEHON,† M. AM. SOC. C. E.

The writer assumes the only substantial justification for a deep waterway connecting the Great Lakes with the salt seas is the deep penetration of the continent to Duluth, Port Arthur, and Chicago. The navigable right of way of the whole Great Lakes System, therefore, is involved in the discussion.

The 1924 project-drafts in Lakes Erie, Michigan-Huron, and Superior with the connecting rivers, are 20 ft. The waterway to the sea contemplates initially 25-ft. and ultimately 30-ft. drafts.

The St. Lawrence River is the geological avenue to the sea through which outflow the surplus waters of the Great Lakes. This river is, therefore, the natural avenue for a navigable waterway to the sea. The water itself, with a volume of 220 000 sec-ft. and a descent of over 220 ft. from the level of Lake Ontario to Montreal, is one of the great power resources of the world. The writer assumes that the development of water power co-ordinately develops and subsidizes a navigable waterway.

This paper describes the Port of Duluth and the various navigable ways through Lake Superior, St. Marys River, Lake Huron, the St. Clair and Detroit Rivers, Lake Erie, through the Welland Canal descending over the Niagara Escarpment 326 ft. to the level of Lake Ontario, and then through Ontario and the Upper St. Lawrence to the Rapids Section of this river. This section, beginning at the Galops Rapids and terminating at the foot of the Lachine Rapids at Montreal, is the locus of the great constructions contemplated at an initial cost exceeding $250 000 000. Little detailed description of the development of the Rapids Section contemplated in the report of the International Joint Commission of 1921 is embodied; but some general principles involved in the best co-ordination of navigation and water power are suggested.

A new International Board of Engineers is studying the report of the Government engineers of 1921, having in mind a review of the earlier conclusions. It has seemed appropriate, therefore, to devote the major portion of the paper to those elements of the problem not within the purview of the report.

The water supply tributary to the St. Lawrence is fully discussed and works are suggested to increase the dependable volume of flow.

* Presented at the Fall Meeting of the Society, Detroit, Mich., October 23, 1924. Discussion on the paper will be closed in **December, 1925**, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

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Two routes from the Great Lakes to the sea investigated by the Board of Engineers on Deep Waterways in 1900, are touched upon. The route by the way of Oswego and the Mohawk to the Hudson, thence to the Port of New York, was preferred in 1900. Since that time, the near completion of the Welland Canal and the desirability of building at least the International Section of the St. Lawrence River Waterway, make preferable a route to the sea through the St. Lawrence River to the foot of Lake St. Francis, thence descending to Lake Champlain and through the Hudson River to the Atlantic.

It is shown that the deeper drafts in the Great Lakes necessary for a waterway to the sea, are also economically desirable for the immense existing internal navigation. It is shown that regulating works, in addition to those already built at the foot of Lake Superior, may be constructed at the head of the St. Clair and Niagara Rivers. These works will raise the lake levels and diminish the excavation needed for 25-ft. drafts.

The regulating works recommended will convert Lakes Superior and Michigan-Huron into vast storage reservoirs as aids to water power in the Niagara and St. Lawrence Rivers, stabilizing the flow and adding 20 000 sec-ft. to the volume of prime flow. The fact is emphasized that in a state of Nature it takes several years for available storage water in Lake Superior to be transmitted for power use in the Niagara and St. Lawrence Rivers; whereas with a complete series of manually controlled regulating works the equivalent of Lake Superior water may be instantaneously made available in these great power streams. And conversely the equivalent of excess water supply in one of the lower lakes, Ontario, for example, may be instantaneously transmitted to the Superior Reservoir or to the Michigan-Huron Reservoir.

Water power at Niagara Falls is touched upon as competitive in a limited market with St. Lawrence power.

The international aspects of the various constructions are discussed, and certain suggestions made for desirable elements to be incorporated in a new Boundary Waters Treaty between Great Britain and the United States.

The St. Lawrence Waterway to the Sea may be narrowly visualized as the St. Lawrence River improved for 25 or 30-ft. navigation between the smooth, stately river at Montreal, and the smooth, stately river above the Galops Rapids. In the intervening 115 miles, the river descends roundly 224 ft. and has a normal flow of 230 000 cu. ft. per sec. Under average river-stage conditions, ocean liners of 30-ft. draft penetrate the continent as far as Montreal—1 000 miles inland from the open Atlantic—making Montreal one of the great ports of the world. Advancing this inland travel of ocean liners to Toronto, near the west end of Lake Ontario, serves no adequate navigational purpose. The Great Lakes still reach nearly 1 000 miles inland after climbing, by locks and canals, 326 ft. up and over the Niagara Escarpment and entering the foot of Lake Erie. The east end of Lake Erie, with Buffalo as its principal port, is really the water origin for present west-bound commerce and the water terminal for east-bound cargoes.

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In the largest sense the St. Lawrence Waterway to the Sea must be visualized as the right of way leading to the Gulf of St. Lawrence from Duluth and Port Arthur, near the head of Lake Superior, and from Chicago, at the extreme south end of Lake Michigan. Deep penetration of the continent is the only adequate justification for great expenditures in the name of navigation. (See Plate II and Fig. 1.)

The St. Lawrence Waterway to the Sea contemplates deep-draft navigation—perhaps 25-ft. depths first, with 30-ft. drafts some decades later. The present navigation scheme for Lake Erie, and the rivers and lakes westward to Chicago, Duluth, and Port Arthur, is for 20 and 21-ft. depths—20 ft. in sheltered channels with soft bottom, 21 ft. where exposed to the surge of wave action and in rock channels. The Deep Waterway to the Sea, therefore, must include, as an essential part of its project, the securing of better depths in all artificial channels, harbors, canals, and locks which deep-draft vessels may need to traverse in the whole Great Lakes System. These depth betterments will be secured by two methods: First, by the comprehensive method of the maintenance of higher lake and river levels; and, second, by the piecemeal, localized method of dredging in trunk-line channels and canals and in important harbors. In many cases both methods will be necessary—the volume of dredging being diminished by the prism gained by the maintenance of higher water-surface levels.

Near Available Mileage

One of the notable things in this waterway project is the relatively large mileage already available for 25-ft. drafts; the mileage which will become available by the maintenance of higher water-surface levels; and the major projects, small in mileage but large in cost, already advancing toward completion. The project as a whole awaits only the breaking through of a few impediments and the construction of works at the heads of the outflow rivers. The penetration of the principal impediment in the Rapids Section of the St. Lawrence will be eventually subsidized in whole or in part by the water power simultaneously developed.

The mileage of the vessel tracks of the Great Lakes is infinite and the mileage of the principal avenues or trunk lines is very considerable. Confining attention to the trunk line with Duluth as the initial westernmost port of the waterway and a point 1 000 miles below Montreal as the eastern gateway to the ocean, the total trunk-line length is 2 340 miles—with Liverpool still 2 200 miles away across the Atlantic. The mileage available after lake-level maintenance is effective and the Welland Ship Canal completed is given in Table 1.

The mileage requiring improvement, dredging, or cleaning up of obstructions, is about 9½% of the 1 338-mile vessel track from Duluth to Montreal; and about 5½% of the 2 341 miles of trunk-line waterway between Duluth and Belle Isle Straits. Outside of this trunk line is the right of way of the salt seas encircling the globe.

TABLE 1.

Section.	Near available, miles.	Requiring improvement, miles.
Duluth to ship locks, St. Marys River	391	3
Ship locks, St. Marys River, to Detour	30	20
Detour to head of St. Clair River	213	3
St. Clair River	37	3
Lake St. Clair	10	8
Detroit River	16	11
Lake Erie to entrance of Welland Canal	227	3
Welland Ship Canal completed about 1930	26	0
Lake Ontario to Tibbetts Point	156	0
St. Lawrence River to Galops Rapids	66	3
St. Lawrence, Galops Rapids to Montreal	40	72
St. Lawrence, Montreal to Atlantic	1 003	0
Totals	2 215	126

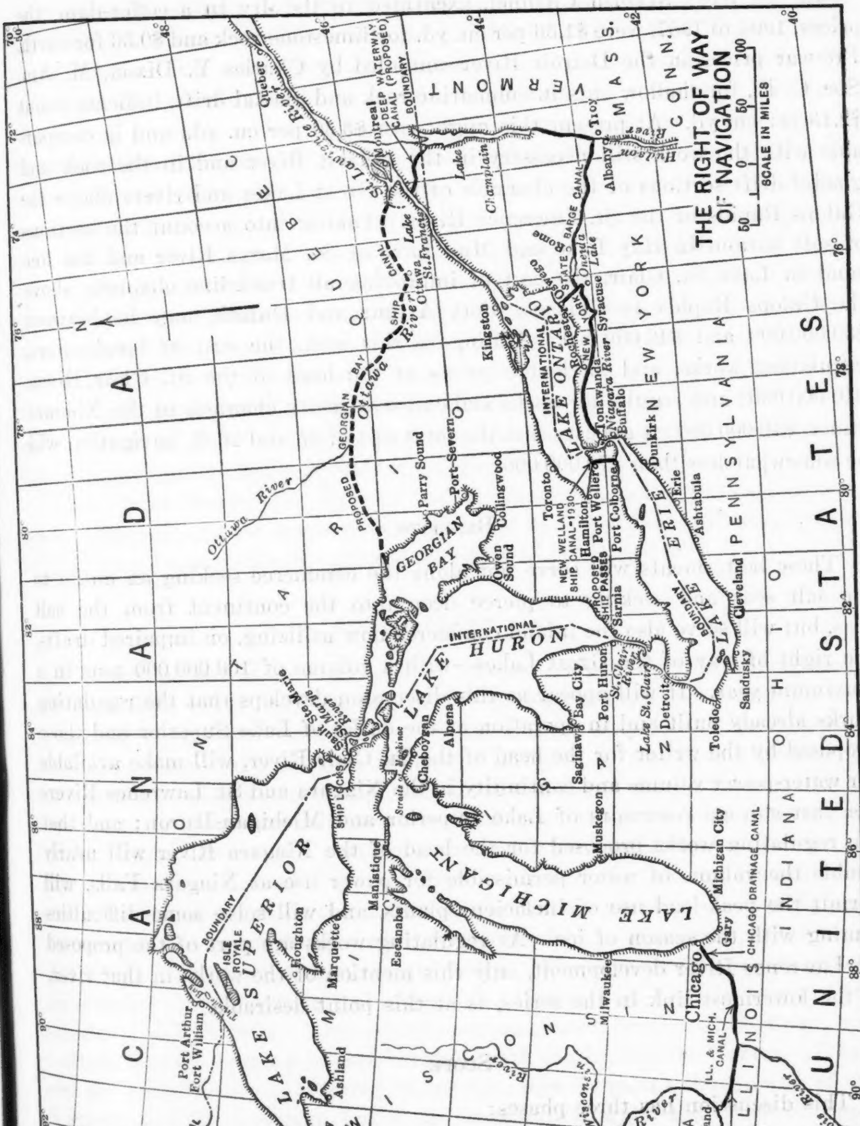
For practical use of the waterway, the mileage of tracks in the principal harbors utilizing deep drafts must be added. Mostly the mileage given in Table 1 and mostly the work in harbors and harbor entrances are in sheltered areas where dredges may work in all weather conditions.

Cost of Improvements

The aggregate yardage to be removed is comparatively small to change 20 and 21-ft. channels, based on extreme unregulated low-water levels of the Great Lakes and interlake rivers, to 25 and 26-ft. channels, based on higher regulated lake and river surface levels. The channel depths which will come with maintained surface levels will be for the most part 22 and 23 ft. between Duluth, Port Arthur, Chicago, Detroit, Buffalo, Toronto, Prescott and Ogdensburg. The remaining 3 ft. of depth for 25 and 26-ft. channels will be secured by cleaning up some of the channels and by dredging to an additional depth of 3 ft. in others. The volume in a mile of prism, 3 ft. thick and 300 ft. wide, is less than 200 000 cu. yd. One-half, at least, of the 50 miles of the Great Lakes is in channels with soft bottoms, as in Lake St. Clair; a large fraction is in glacial drift formations with boulders, as in St. Marys River above the locks; while a comparatively small mileage is in sand rock or limestone formations, as in the Neebish Channels in St. Marys River, in St. Marys Falls Canal and in the Lower Detroit River. Harbor channels are for the most part in soft materials. Only the major harbors of the Great Lakes need improvement for 25 and 26-ft. drafts.

The cost of deepening St. Marys River represents the largest item of dredge work. Louis C. Sabin, M. Am. Soc. C. E., U. S. Civilian Engineer in charge of the construction, maintenance and operation of the locks, canals and channels of St. Marys River, has provided the following unit prices for submarine excavation:

	Pre-war.	1924.
Earth per cubic yard	\$0.11 to \$0.15	\$0.18 to \$0.30
Medium rock per cubic yard	2.00	3.00
Hard rock per cubic yard	3.00	4.00 to 4.50


THE RIGHT OF WAY OF NAVIGATION
SCALE IN MILES
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U N I T E D S T A T E S
LAKE SUPERIOR
LAKE MICHIGAN
LAKE HURON
GEORGIAN BAY
LAKE ERIE
LAKE ONTARIO
Lake Champlain
NEW WELLAND SHIP CANAL-1930
PROPOSED SHIP PASSES
PROPOSED GEORGIAN BAY SHIP CANAL
DEEP WATERWAY CANAL (PROPOSED)
STATE BARGE CANAL
CHICAGO DRAINAGE CANAL
ILL. & MICH. CANAL
HENNEPIN CANAL
INTERNATIONAL BOUNDARY

FIG. 1.

In the West Neebish Channel, excavated in the dry in a coffer-dam, the prices, 1904 to 1907, were $1.36 per cu. yd. for limestone rock and $0.56 for earth. Pre-war prices in the Detroit River compiled by Charles Y. Dixon, M. Am. Soc. C. E., for shallow cuts in submarine rock and glacial drift, indicate about $2.15 per cu. yd. At present this might cost $3.50 per cu. yd., and is comparable with the work still necessary in the Detroit River and in the rock and glacial drift sections of the channels of the Great Lakes and rivers above the Galops Rapids of the St. Lawrence River. Taking into account the sections of soft bottom in Hay Lake and Mud Lake of St. Marys River and the fine sand in Lake St. Clair, the cost of improving all trunk-line channels above the Galops Rapids to Chicago, Port Arthur and Duluth, may be between $30 000 000 and $40 000 000. Adding to this sum, the cost of breakwaters, regulating works, and navigable passes at the head of the St. Clair River, $10 000 000; and regulating works and excess-capacity channels in the Niagara River, $10 000 000; it appears that the total cost of 25 and 26-ft. navigation will be somewhat less than $70 000 000.

Benefits

These betterments will serve not alone the commerce seeking an outlet to the salt seas and seeking to pierce deep into the continent from the salt seas, but will serve also the inland commerce now utilizing, on impaired drafts, the right of way of the Great Lakes—with a volume of 100 000 000 tons in a maximum year. It will appear as this discussion develops that the regulating works already built and in operation at the outlet of Lake Superior and those proposed by the writer for the head of the St. Clair River, will make available for water-power volume and continuity in the Niagara and St. Lawrence Rivers the vast storage reservoirs of Lake Superior and Michigan-Huron; and that the regulating works proposed for the head of the Niagara River will nearly double the volume of water permissible for power use at Niagara Falls, will permit the peak-load use of inefficient plants, and will solve some difficulties coming with the season of ice. As regulating works are part of the proposed St. Lawrence River development, only this mention of the works in that river, as the lowermost link in the series, is at this point desirable.

Scope

This discussion has three phases:

First.—The internal commerce of the Great Lakes as it now exists as an economic asset of two nations, with the physical limitations which handicap it, and the proposed engineering remedies.

Second.—The outlets to the salt seas existing and contemplated, including the St. Lawrence avenue.

Third.—Water power at Niagara Falls and on the St. Lawrence River, as an economic asset of two nations, with the physical and international elements which handicap it—and the engineering and international remedies.

It appears obvious that if an expenditure of $70 000 000 in the Great Lakes System above the Galops Rapids, in addition to the New Welland Ship Canal now nearing completion, did not serve as a paying investment for the movement of the 100 000 000 tons of fresh-water cargoes, it might be a venture hinging on a problematical salt-water commerce; and if the expenditure of $200 000 000 or $300 000 000 to make possible a problematical volume of commerce between fresh water and salt water, did not co-ordinately bring into use upward of 4 000 000 e.h.p. to enrich two nations, the St. Lawrence avenue to the sea might be forgotten for a generation or more. If, however, the expenditure of $70 000 000 in the Great Lakes System above the Galops Rapids serves economically the fresh-water commerce of the United States and Canada and, at the same time, vitalizes and multiplies the water power at Niagara Falls and on the St. Lawrence River, it ceases to be a doubtful venture; and if the water-power development of the St. Lawrence subsidizes the cost of the navigable avenue of this river above Montreal, the St. Lawrence Waterway to the Sea will not need to be forgotten for a generation.

These three things—the right of way of the Great Lakes, the various avenues to the sea, and the water power generated by the excess outflowing waters of this vast drainage basin—are the elements to be discussed.

Standards

The right of way of the Great Lakes, between Duluth, Port Arthur, Chicago and Lake Erie ports, with Buffalo as the present easternmost terminal, will be touched upon first. Before doing so, however, it may be well to state that the subject matter of this paper deals with projects so vast that $1 000 000 is the lowest unit to be given consideration. If this is not clearly apprehended at the outset, the large expenditures suggested may be set aside as extravagant and visionary. It is well, also, to state that where estimates of the cost of improvements recommended by the writer are given, they are not true detailed estimates, but are more in the nature of illuminated guesses. It is well, also, to state that the ton of freight wherever referred to in this paper means a ton of 2 000 lb. Miles are always statute miles. Wherever levels are mentioned, the reference is to height in feet above mean tide at New York in terms of the adjusted levels of 1903. This adjustment was made by the U. S. Coast and Geodetic Survey working in collaboration with the U. S. Lake Survey. Wherever the Lake Survey is referred to in this paper it means the Survey of the Northern and Northwestern Lakes, an organization under the Corps of Engineers, U. S. Army, with headquarters at Detroit, and with jurisdiction for surveys, chart-making and hydraulic and other investigations covering the whole Great Lakes System, extending westward and including the Lake of the Woods and eastward and southward to include Lake Champlain and connecting waterways.

The writer has mentioned casually the expenditure of something approaching $70 000 000 within the Great Lakes System to secure 25 and 26-ft. navigation. The United States Government has already expended in developing the

right of way of the Great Lakes, including channels, canals, ship locks, breakwaters, harbors, light-houses, and aids to navigation, upward of \$150 000 000. The New Welland Ship Canal, now under construction, to be completed perhaps by 1930, will cost fully \$100 000 000. The Engineers' report for the St. Lawrence development between a point a little above the Galops Rapids and Montreal is over \$250 000 000. It is probable that any of these figures may vary in ultimate estimates by 10% or more, with a possibility always that the variation may be upward.

Navigation Values

To make understandable and justifiable proposed expenditures comparable with the cost of the Panama Canal, a quotation is made from the Presidential Address of the late Alfred Noble, Past-President, Am. Soc. C. E., for many years Dean of American engineers. This address,* entitled "The Development of the Commerce of the Great Lakes," was delivered before the Society in 1903.

"This wonderful development of a few leading articles of freight has resulted in cheaper transportation than known elsewhere, except on long ocean routes. The most reliable and complete data existing are obtained at the St. Mary's Falls Canals by the co-operation of the Governments of Canada and the United States. The average freight rate on all classes of freight during 1902 was 0.89 mill per ton-mile, the average haul being 827.4 miles. If the freight had been carried by rail the average rate could hardly have been less than 3 mills per ton-mile, and the average haul would have been about the same as by water. On this basis the saving in transportation by the water route during one year was nearly \$63 000 000. This amount of saving on Lake Superior commerce alone, during a single year, is within \$5 000 000 of the entire amount appropriated by the United States for all harbors and waterways on the Lakes above Niagara Falls from the formation of the Government; if the commerce between Lake Michigan and Lake Erie be included, the annual saving greatly exceeds the amount thus appropriated. Indeed, the saving during one year on a single article, iron ore, would repay all the money expended up to this time on the waterways and harbors which it traverses on its way from mine to furnace. Viewed in this way, no great investment was ever made by a corporation or government with such a magnificent showing of profit. But a far broader view may be taken justly; this water route has not merely afforded a saving in expense of transportation, but has been creative of a large part of the wealth of the nation. Without it the development of our manufactures of iron and steel and other industries dependent on them or related to them, all constituting a great share of the national wealth, would have been retarded or impracticable; without it the prairies of the Northwest would have remained unproductive and unpeopled for a much longer time. With these broad considerations in view we may place to the credit of Lake transportation a sum perhaps hardly definable, but compared to which the largest above mentioned is a mere trifle."

In the year 1902, referred to by Mr. Noble, the commerce through the Soo Locks amounted to about 36 000 000 tons with a valuation of a little over \$358 000 000. During the same year, the estimated freight passing through the Detroit River was over 44 000 000 tons with an estimated value of nearly \$441 000 000. Fourteen years later, in 1916—a year when commerce

* *Transactions*, Am. Soc. C. E., Vol. L (1903), p. 327.

was stimulated by the World War—the freight passing the Soo Locks was nearly 92 000 000 tons and that through the Detroit River exceeded 100 000 000 tons, with a valuation of nearly $1 070 000 000.

Applying the same rule for estimating the economic value of the right of way of the Great Lakes as used by Mr. Noble, the saving for the single year, 1916, was $175 000 000 for commerce passing the Soo Locks.

Water Values

It will appear later that the outflowing waters of the Great Lakes have incredibly large values. It has been estimated that the diversion of 10 000 cu. ft. per sec. in the Drainage Canal, is worth $8 000 000 a year to Chicago. The total ultimate usable power in the Niagara River is nearly 3 000 000 e.h.p. Should the installations for the use of this power cost $100 per e.h.p.—a low figure—the investment in that river alone will be $300 000 000. The present capitalization of the Niagara Falls Power Company is more than $75 000 000. On the St. Lawrence River upward of 4 000 000 e.h.p. will ultimately be developed. This will warrant an expenditure of more than $400 000 000.

Justification

After these vast sums are fully comprehended and digested, the justification of the expenditure of considerable sums in securing the highest carrying capacity of the vessels of the Great Lakes, through better and deeper channels —and, hence, cheaper freight rates—becomes apparent; and the advisability of utilizing for water-power betterment the vast storage reservoirs of Lake Superior and Lakes Michigan-Huron becomes obvious.

A great temptation exists for the writer to go into the historical phase of the development of the commerce of the Great Lakes. Instead, reference is again made to the Presidential Address of the late Mr. Noble previously noted. The beginnings of the navigation of the Great Lakes reach back to a period of exploration, adventure and romance, perhaps more interesting than the cold recital of statistical facts and the cold development of engineering projects to add to the economic value of the right of way of the Great Lakes. This paper deals with the present and with the future and utilizes the past only to make the future understandable.

During the 1923 season of navigation—which may be regarded normally as having a duration of a little less than eight months, from April 15 to December 15—the freight passing the locks at Sault Ste. Marie amounted to over 91 000 000 tons, with a value exceeding $1 000 000 000. The total freight paid was nearly $81 000 000, which included loading and unloading, except in the case of coal. The average distance carried was roundly 800 miles. The average freight charge per ton for this distance was 88 cents, which indicates an average cost of 1.1 mills per ton-mile. It is a fair assumption that this freight if carried by rail would have cost over four times as much.

Deferred Maintenance

This land-locked waterway—the right of way of the Great Lakes—should be maintained in the same excellent condition as the right of way of a great

railway, such as the Pennsylvania System or the New York Central Lines; but the fact is that the levels of the Great Lakes, on which the drafts of vessels depend for their loading capacity, have been permitted to lower, following the caprice of natural rainfall conditions; and have been artificially lowered by diversions for sanitation at Chicago, by diversions for power and navigation purposes in the Welland Canal and in the Niagara River, and have been lowered by the additional outflow capacity of the rivers themselves, created by the deepening of river reaches for ship channels. The depth betterment, for 20 and 21-ft. channels between Buffalo, Chicago, and Sault Ste. Marie, has been secured by excavating the lake and river bottoms below the level of extreme low water rather than by the elimination of extreme low water through the maintenance of the surface levels of the lakes.

The first step toward the St. Lawrence Waterway to the Sea is to bring up the levels of the lakes and the inter-lake rivers between the foot of Lake Erie, the southernmost extremity of Lake Michigan, and the westernmost extremity of Lake Superior. The maintenance of the surface levels of Lake Ontario is an integral part of the St. Lawrence River project and will be discussed in the general review of that project as part of the St. Lawrence Waterway to the Sea.

During the present season of navigation (1924), the St. Marys River, below the foot of the Soo Locks, and Lakes Michigan-Huron have been at their lowest recorded stage in a period of 65 years. The reason for this low stage involves the various diversions already mentioned, but is caused predominantly by the temporary absence of abundant rainfall in the Great Lakes System and by the absence of regulating works at the head of the St. Clair River, where the lake waters outflow toward Lake St. Clair, the Detroit River and Lake Erie. (See Plate III.)

It must be understood that the dearth of precipitation shown by the low levels of Lakes Michigan-Huron and Erie is not confined to this drainage basin but is widespread over the United States. California is having a water famine without precedent. The Mississippi River at Minneapolis in January, 1924, showed its lowest recorded flow. Most frequently, precipitation meagerness is somewhat localized. The drainage basin of Lake Superior may have an abundant water supply at a time when the Lake Erie Basin is deficient. It will appear later that the diversity factor applies to the basins of the Great Lakes and that advantage of this factor may be secured only by two operations: First, by conserving the flood waters in the upper reservoirs; and, second, by arranging for the transfer of these stored waters without time loss to the lake where navigation or water power requires additional supply. This obviously is the function of regulating works which will be discussed in detail in subsequent paragraphs.

Value of Draft

It is desirable to apprehend clearly what draft means in the carrying capacity of the freighters of the Great Lakes. The largest of these freighters

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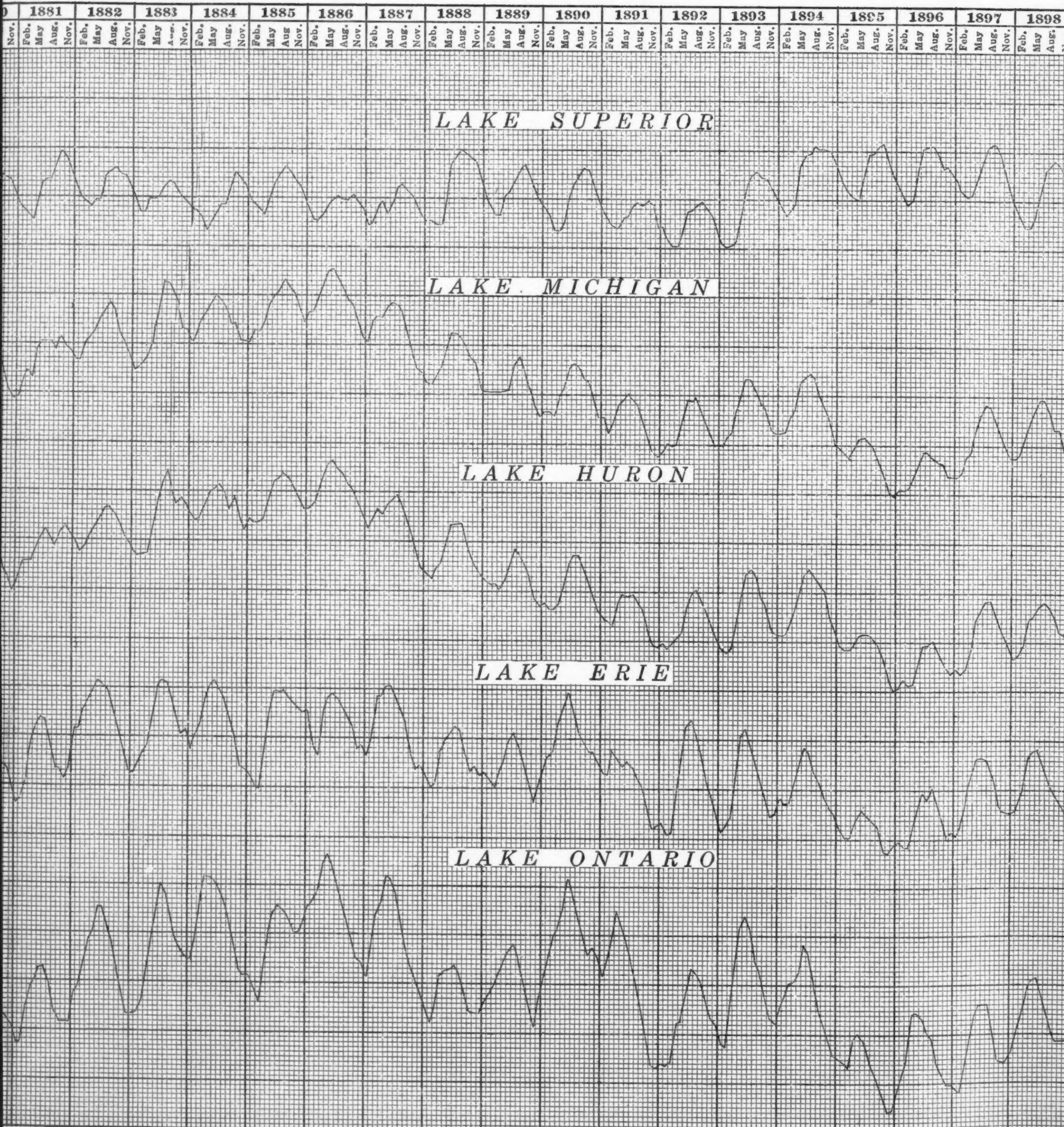
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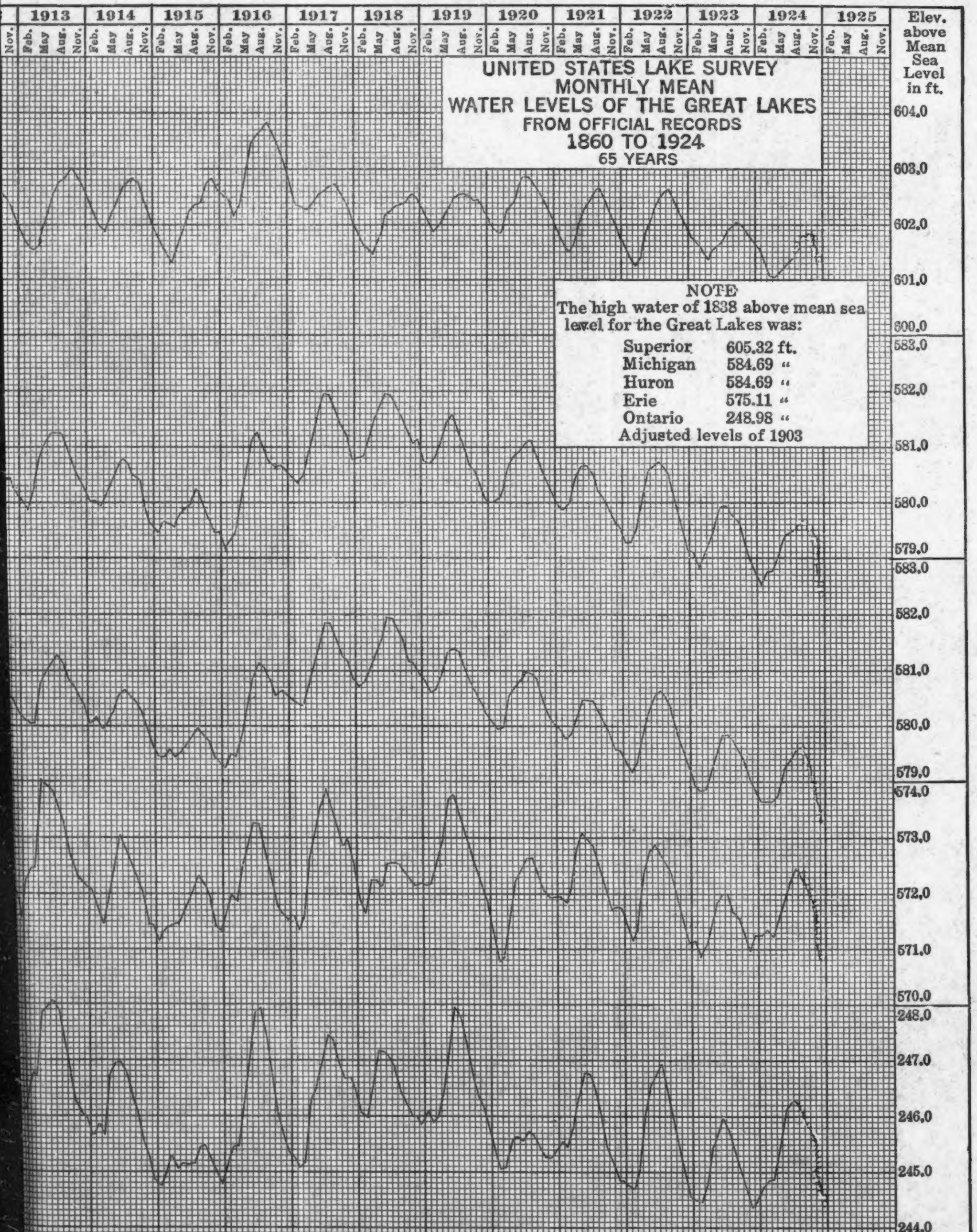
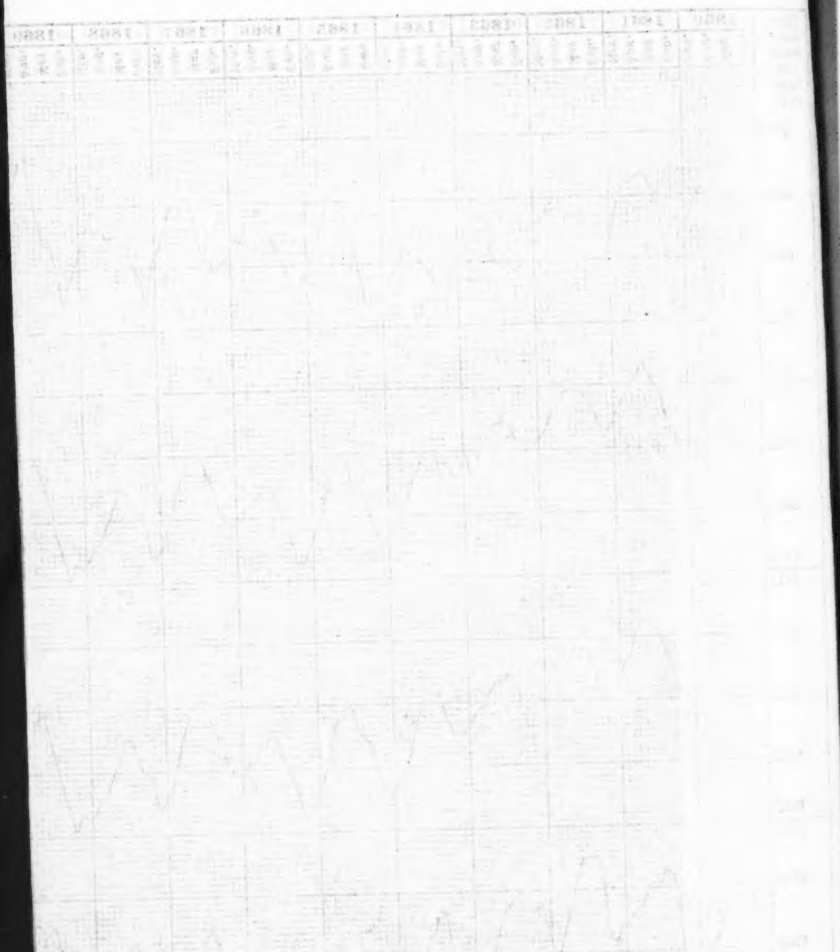
PLATE III.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1925
SHENEHON ON
THE ST. LAWRENCE WATERWAY
TO THE SEA





exceeds 600 ft. in length. The tonnage of freight for a vessel drawing 20 ft. of water is carried by a displacement of perhaps 10 ft., the weight of the vessel and its non-productive cargo requiring the displacement of the other 10 ft. This means for this type of vessels that a loss of a foot in draft means a loss of upward of 1 000 tons of freight.

The elevation of Lakes Michigan-Huron for the season of navigation of 1924 is 579.3. The mean stage of Lakes Michigan-Huron during the seasons of navigation over a period of 65 years was 581.5. Based on the mean elevation for a period of 65 years, this indicates that in 1924 Lakes Michigan-Huron are deficient in navigable depth 2.2 ft., which means a loss of 2 200 tons in carrying capacity for each large lake freighter on each trip, with little change in the cost of operation. It is obvious that this must mean increased freight rates.

Expressing the loss of draft in inches as a matter of convenience, the season of navigation shows a deficiency in draft of 26 in. Chicago is chargeable with 5 in., through its diversion to the present time of about 9 000 cu. ft. per sec. Diversions in Lake Erie for purposes of navigation and water power are chargeable with perhaps a back-water reflection of 1½ in. Enlargements in the channels of the St. Clair and Detroit Rivers are responsible for perhaps 3½ in. The remaining loss of draft of 16 in. is chargeable to lack of normal precipitation and the absence of regulating works at the head of the St. Clair River.

The economic value of an inch in draft has been estimated as $500 000 a year. While this figure may be a little on the large side, it will be used in this paper. It must be understood that the large types of freighters on the Great Lakes are capable of loading to 24 ft., that they load to the last inch available in the channels they are to traverse, and that the loading is directed by telegraphic advice.

Assuming the correctness of this figure of $500 000 for an inch draft, the loss for the 1924 season of navigation, based on 26 in. of deficiency, is $13 000 000. With reasonable reductions for a conservative estimate, it is certain that this loss under ordinary trade conditions would mean $10 000 000 for the navigable season of 1924. It happens, however, that depressed conditions have resulted in smaller shipments than usual and that vessels are available to carry the tonnage to be moved on lesser drafts. For this reason, the real economic loss on the Great Lakes for 1924 is somewhat less than the figure given.

During the season of 1923, the elevation of Lakes Michigan-Huron was 579.7. Based on a mean water-surface elevation for these lakes of 581.5, the deficient draft for this season of heavy shipments was 22 in., with a loss of upward of $10 000 000 for the year.

Assuming that these estimates of economic loss coming from deficient drafts in the channels of the St. Marys River, Lakes Michigan-Huron, and in Lake Erie, are within gunshot of correct, it appears that the maintenance of lake levels by regulating works at the head of the St. Clair River, installed at the cost of perhaps $10 000 000, and regulating works at the head of the

Niagara River at a cost of perhaps \$10 000 000, is a most obvious economic necessity; and if it appears that some part of the cost of these works may be properly charged to water-power betterment at Niagara Falls and in the St. Lawrence River, the economic desirability is still further emphasized.

Low Levels Anticipated

The degraded levels of Lakes Michigan-Huron in the years 1923 and 1924, which have been cited, were a reasonable expectancy based on the water-supply conditions of the Great Lakes over a period of 65 years beginning in 1860. In the year 1895, the level of Lakes Michigan-Huron during the season of navigation was only an inch higher than in 1923; and in the year 1896, the level of these lakes was the same as in the season of 1923. These low levels of more than a quarter of a century ago preceded the deeper excavations in the St. Clair and Detroit Rivers, preceded the opening of the Chicago Drainage Canal and preceded any large diversions for water-power purposes in the Welland Canal and in the Niagara River.

Desirable Elevation

During the seasons of navigation for a 5-year period, 1883 to 1887, inclusive, the elevation of Lakes Michigan-Huron was 582.8. The writer suggests the regulation of these lakes during the season of navigation at the same elevation, 582.8, with a possible pull-down of 1 ft. of storage for water-power betterment in the Niagara and St. Lawrence Rivers in periods of meager precipitation.

Assuming the normal regulated stage of Lakes Michigan-Huron as 582.8 during the season of navigation, the deficient drafts during the season of 1923 amounted to 37 in. and during the season of navigation of 1924 amount to 42 in. These figures may be lessened to 25 and 30 in., based on the lower regulated level of these lakes after an occasional pull-down of 1 ft. for the use of the accumulated storage.

Conditions at Soo Locks

The effect of these low levels, and particularly that of 1924, in the lower approach to the ship locks at Sault Ste. Marie, should be noted. The water-surface elevation in the river just below the locks in a period of small river flow, such as that at the present time, is about 9 in. above the water-surface elevation of Lakes Michigan-Huron. In times of ample flow, the water level at the lower entrance to the locks may be 1 ft. or 1½ ft., under present conditions, above the level of these lakes. With the present degraded mean levels for the season of navigation of 1924, the permissible draft of vessels passing through the Poe Lock is about 17.5 ft. and the draft of vessels passing the Canadian Lock is limited to 17.7 ft. These locks were built in connection with the project in which the 20 and 21-ft. drafts provided for, were based on mean lake levels rather than on extreme low lake levels. It is for this

reason, and because the level of the St. Marys River at the lower approach to the locks has been further degraded by excavating deeper channels throughout the lower river, that these two ship locks are of no value this year (1924) for other than light-draft vessels. Vessels loading to 18 or 19 ft. must pass through the two new American locks—the Davis Lock and the Sabin Lock. It is obvious that bringing Lakes Michigan-Huron levels up 3 ft. or more by regulation, will make available again these two ship locks for 20-ft. navigation.

During the present season, the conditions in St. Marys River and in Lakes Michigan-Huron control drafts. During other seasons, the drafts of vessels have been limited by Lake Erie channels, which will be discussed later in connection with regulation works at the head of the Niagara River.

THE PORT OF DULUTH

The zero point for the St. Lawrence Waterway to the Sea is at Duluth, at the westernmost extremity of Lake Superior, 950 miles westward and 410 miles northward from the Port of New York and 1 420 miles eastward from Seattle. (See Plate II.) By a somewhat remarkable natural coincidence, the St. Louis River enters Lake Superior at this westernmost point and a delta formation of alluvial material forms a natural breakwater 8 miles in length, protecting the estuary of the river from storms. This spit or breakwater is pierced by harbor entrances in two places, one toward its northwestern extremity leading to the inner basin of the Harbor of Duluth. Southeasterly from this entrance about 6¼ miles is an entry first, through artificial breakwaters to an outer harbor, and, then, through a passage 400 ft. wide, to the inner basin of Superior Harbor. As the St. Louis River forms the boundary line between Minnesota and Wisconsin, the twin harbors connected by a vessel channel behind the breakwater, are interstate. Aside from the 4-mile channel connecting the two basins, there are more than 13 miles of channels leading to the various docks of the two harbors.

The harbor entrances are already deep enough for 25-ft. navigation. For the most part, the various channels and the basins of the harbor will require dredging to a depth of perhaps 3 ft. to provide for 25-ft. navigation, with higher regulated levels of Lake Superior. The material to be excavated in deepening the harbor channels is largely silt and sand, underlain by boulder clay. All the material may be handled by suction dredges, using cutters in the clay, at a probable cost not exceeding 20 cents per cu. yd.

The Government has already spent—or is completing the expenditure of—a little over $8 500 000, including maintenance. The cost of maintenance for 1925 is estimated at $52 500. Private companies and the City of Duluth have expended about $200 000 in breakwaters and trunk-line channels. An expenditure of about $1 000 000 will provide 25 and 26-ft. depths in the twin harbors.

The following statements are briefed from the 1923 reports of E. H. Marks, Major, Corps of Engineers, U. S. A., M. Am. Soc. C. E., in charge of the work:

The entire protected harbor area aggregates 19 sq. miles, with about 49 lin. miles of frontage, only 6 miles of which are occupied by wharves. Nine

iron ore docks, aggregating 8 720 ft. in length, have a total combined storage capacity of 1 080 000 tons; 24 coal docks have an aggregate storage capacity of 11 305 000 tons; 26 elevators have a total storage capacity of 36 325 000 bushels; a modern cement elevator has a storage capacity for 114 000 bbl. of cement, with an unloading capacity of 1 000 bbl. per hour. Most of these docks and elevators are up to date in facilities for the quick handling of ore, coal, and grain, and are probably not surpassed by those of any other port. Practically all the wharves, docks, and elevators are privately owned. The facilities for handling package freight, lumber and general merchandise were primitive up to 1924, when a new terminal plant, privately owned, but open to the public, was made available. Three shipbuilding yards and forty-three wharves handle freight other than iron ore, coal, and grain. Ten railroads, either direct or through affiliated lines, connect with the water terminals.

Major Marks sums up the value of improvements in the Duluth and Superior Harbor as follows:

"The effect of the work done under the existing and previous projects has been to facilitate navigation by means of deeper and wider channels, with commodious anchorages and turning basins and safe entrances to the harbor, as well as to give refuge to vessels seeking shelter from storms. Through these improvements, which permit the use of larger vessels, freight rates have been materially reduced."

Inter-lake navigation, based on records for the past 20 years, begins normally on April 20, and terminates on December 14, showing a navigation season of 239 days for Duluth.

Duluth Commerce

In 1923, the commerce of the Duluth-Superior Port shows over 59 000 000 tons, with a valuation of nearly $450 000 000. Of this commerce, the outbound tonnage amounted to a little over 45 000 000, while the inbound commerce amounted to a little over 14 000 000 tons. This shows an outbound balance of roundly 31 000 000 tons, indicating that many upbound vessels travel light or with water ballast. Iron ore, with a value of $4.32 per ton, constitutes 58% of the outbound valuation and 94% of the outbound tonnage. Wheat, high in value, but low in weight, constitutes 13% in value and 7½% in tonnage. Shipments of animals and animal products show a valuation of $14 500 000; vegetable and food products, including wheat, $17 500 000; wood and wood products, $800 000; unclassified freight, $3 900 000; and copper nearly $8 000 000. The dominant inbound cargo is coal, 12 700 000 tons in 1923, with a value of $72 245 000. The total inbound commerce has a valuation of nearly $132 000 000.

Canadian Ports

About 200 miles northeast of Duluth are the harbors of Port Arthur and Fort William in Thunder Bay. These are the terminals for the railways that tap the fields of Canada's great western granary and bring wood products as well. This harbor is landlocked against Lake Superior seas by Isle Royal lying across the entrance to Thunder Bay, by Pie Island lying midway of the

entrance, and by Thunder Cape. This westernmost Canadian port, with its capacious roadstead, has unlimited capacity and ample drafts for ocean vessels.

Type of Vessel

The largest portion of the commerce of these ports is carried in a typical vessel of a length of 600 ft., with a beam of 60 ft., and built for drafts of 24 ft. The engine is carried aft, and the freight hold extends nearly the full length of the vessel. The deck is largely made up of hatches for rapid loading and unloading. The economic speed of travel in open water is about 11 miles per hour. The engine has about 3 000 i.h.p.

Duluth to the Soo

One of these freighters, carrying 10 000 tons of cargo on a 20-ft. draft, leaving Duluth, traverses the deep waterway of Lake Superior—1 000 ft. deep in places—and reaches the ship locks at Sault Ste. Marie, 394 miles away, in about 36 hours. It requires an hour for the freighter to pass the ship locks, descending 20 ft. to the level of St. Marys River below the rapids.

The Sault de Sainte Marie

Fig. 2 is an airplane view of the layout of the various works at Sault Ste. Marie, and Fig. 4 explains the photograph and gives much desirable information.

It should be noted that regulating works, consisting of a battery of Stoney gates, control the outflow and levels of Lake Superior. These regulating works and St. Marys Falls Canal are about 14 miles down stream from the foot of Lake Superior. Upper St. Marys River is for the most part a mile or more in width and runs deep and still. The descent from Lake Superior level to the level of the river at the regulating works varies somewhat with the volume of flow, but may be taken as 0.3 ft.

The writer suggests the regulation of Lake Superior at Elevation 603 as a mean during the season of navigation, with a pull-down of 1 ft. for storage water in case of navigation need in the lower lakes or in the water powers of the Niagara and St. Lawrence Rivers. Based on these regulated lake surfaces, about 3 miles of dredging—deepening 22-ft. channels to 25-ft. channels—would be required in the upper river.

Ship Locks

Four ship locks on the American side of the river and one ship lock on the Canadian side are in operation. Down-bound vessels approaching these locks pass through about a mile of canal with ample mooring space. The Canadian Lock, completed in 1895, is 900 ft. long and 60 ft. wide, and is intended for 22-ft. drafts, but during the mid-season of 1924, due to depressed river pool conditions, low lake levels, and small volume of river outflow, the draft over the limiting barrier at the lower entrance of the locks was only about 17.7 ft. On the American side, the Weitzel Lock, 515 ft. long and 80 ft. wide, narrowing at the gates to 60 ft., was completed in 1881 and was in-

tended for 14-ft. navigation, but has only about 12 ft. over the lower breast-wall. The Poe Lock is 800 ft. long and 100 ft. wide, built for 22-ft. drafts, but in 1924 had about 17.5 ft. in the lower entrance. The writer had a small part in the building of this ship lock. It was intended as a 4-vessel lock, each vessel having a length of 400 ft. and a beam of 50 ft. It was completed in 1896 and in the interval of less than 30 years, it has become a single-vessel lock for a typical lake freighter when traveling light. The Davis Lock was completed in 1914, has a 1 350-ft. length and an 80-ft. width, with 24 ft. of water on the lower breast-wall despite lower-pool depression, small outflow, and low lake levels. The Sabin Lock (Fig. 3), of the same dimensions as the Davis Lock, was opened to traffic in September, 1919. It is in gracious appreciation that this lock bears the name of Mr. Louis C. Sabin, in local charge of construction and operation of the works in St. Marys River since 1906. The other American locks bear the names of officers of the Corps of Engineers. (See Fig. 2.)

The Poe Lock had a pre-war cost of \$3 000 000 and the Sabin Lock, built in 1913 to 1919, cost \$2 500 000. The total cost of improvements in St. Marys River to July, 1923, aside from the practically obsolete Weitzel Lock, amounts to \$30 000 000. The cost of operation of the locks and canals on the American side was \$141 200 in 1923 and the maintenance cost was, roundly, \$105 000. The total cost of operation and maintenance during the past 10 years has amounted to $2\frac{3}{4}$ mills for each ton of cargo passing the ship locks.

It should be noted that the necessary time spent by vessels in traversing 1.6 miles of canal and passing the ship locks, averages roundly $1\frac{1}{2}$ hours. This means that practically 1 hour is necessarily spent in the lockage. This time is in part utilized in taking on supplies.

With regulated Lake Superior and Lakes Michigan-Huron levels, the Davis and the Sabin Locks will become available for 25 and 26-ft. navigation, although this will probably involve a lowering by perhaps 1 ft. of the breast-wall and the miter-sills of the upper approach. It should be accentuated that increased drafts and larger cargoes mean fewer vessels and, therefore, fewer necessary locks. The Canadian Lock and the Poe Lock will still normally remain available for 20-ft. navigation, while the two new American locks will care for deep-draft vessels. This link in the St. Lawrence Waterway from Duluth to the Soo is, therefore, substantially existent.

St. Marys Falls Canal Statistics

An average of 92 vessels per day passed the five locks during the season of 1923, of which 55 passed through the Davis and Sabin Locks and 17 through the Canadian Lock.

In 1923, the total freight carried was 91 380 000 tons. This freight had a valuation of \$1 026 045 064, and the freight charges amounted to \$80 843 000. The east-bound freight passing through the Soo Locks was 71 236 000 tons, and the west-bound, 20 144 000 tons. The valuation of 774 registered vessels using the locks was \$254 630 000. The average distance which freight was carried was 801.3 miles and the average cost per ton for freight transportation

FIG. 2.—AIRPLANE VIEW OF WORKS OF NAVIGATION AND WATER POWER AT SAULT STE. MARIE.

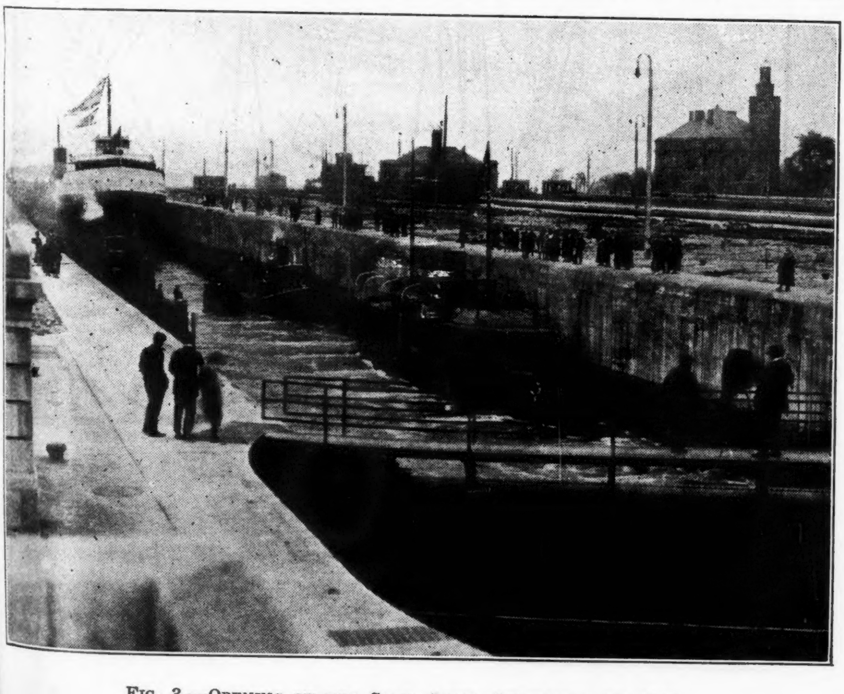


FIG. 3.—OPENING OF THE SABIN LOCK, SEPTEMBER 18, 1919.

Paper

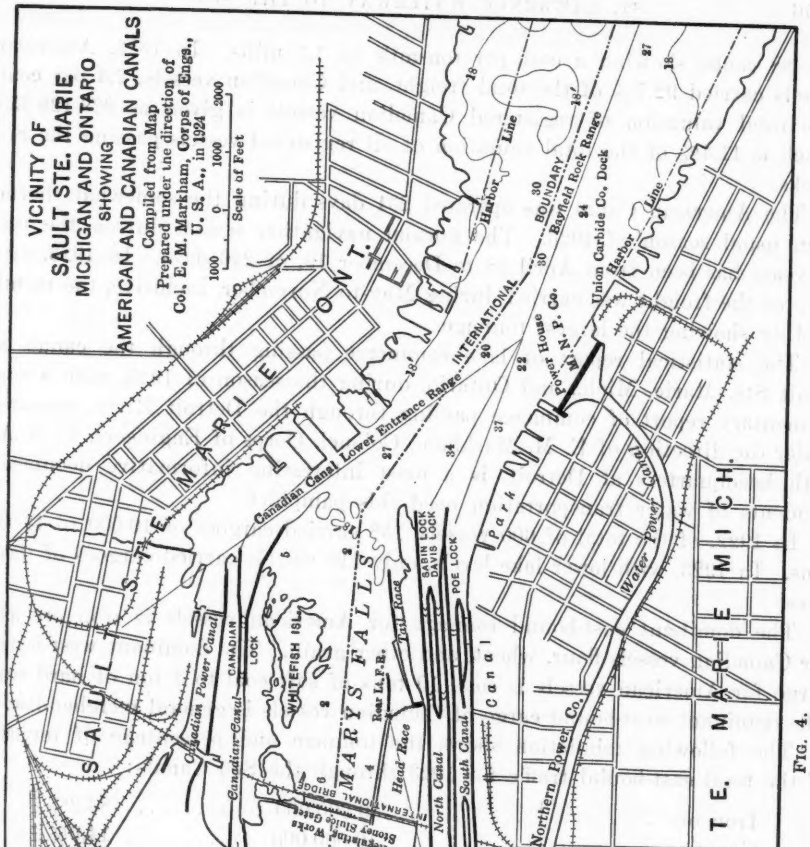

VICINITY OF
SAULT STE. MARIE
MICHIGAN AND ONTARIO
SHOWING
AMERICAN AND CANADIAN CANALS
Compiled from Map
Prepared under the direction of
Col. E. M. Markham, Corps of Engs.,
U. S. A., in 1924
1000 0 1000 2000
Scale of Feet
SAULT STE. MARIE ONT.
SAULT STE. MARIE MICH.
ST. MARYS FALLS
ST. MARYS RIVER
WHITEFISH ISL.
VIDAL SHOALS
Vidal Shoals
Canadian Power Canal
Canadian Canal
CANADIAN LOCK
Canadian Canal Lower Entrance Range
Canadian Canal Upper Entrance Range
INTERNATIONAL BOUNDARY
INTERNATIONAL BRIDGE
Regulating Works
Stoney Sluice Gates
Head Race
Tail Race
Rear L. F. R.
North Canal
South Canal
Canal
SABIN LOCK
DAVIS LOCK
POE LOCK
Park
Water Power Canal
Michigan Northern Power Co.
Power House
M. N. P. Co.
Union Carbide Co. Dock
Bayfield Rock Range
Harbor Line
Channel Range Line
Pittsburgh Coal Co.
Contour
18 Ft.

FIG. 4.

was 88 cents, showing a cost per ton-mile of 1.1 mills. In 1923, American vessels carried 92.7% of the total freight, and Canadian vessels, 7.3 per cent. The total valuation of registered Canadian vessels is given as $29 128 350, which is 11.4% of the total valuation of all registered vessels passing the Soo Locks.

The American Canal was operated 231 days during the somewhat shorter than usual season of 1923. The normal navigation season during the past 20 years has been from April 18 to December 19, or 246 days. In 1923, over 98% of the freight was carried during May to November, inclusive, the month of July showing the largest tonnage.

The statistical report of lake commerce passing through the canals at Sault Ste. Marie, Mich., and Ontario, during the season of 1923, with a supplementary report of commerce passing through the Detroit River, prepared under the direction of E. M. Markham, Colonel, Corps of Engineers, U. S. A., with headquarters at Detroit, is a most interesting informative document. Students of water transportation need this pamphlet.

In 1922, of the total of 600 vessels, 158 carried cargoes of 10 000 to 14 000 tons. In 1923, with lower lake levels, only 110 vessels carried cargoes of these sizes.

The dominant east-bound tonnage for American vessels is iron ore and for Canadian vessels flour, wheat, and other grain. The dominant west-bound cargo for American vessels is coal, 10 tons of soft coal to 1 ton of hard coal. The dominant west-bound cargo of Canadian vessels is general merchandise.

The following tabulation shows the tonnage and percentage of tonnage of the total east-bound traffic for 1923 through the Soo Canals:

Iron ore	59 200 000	tons	83.2%
Grain	10 000 000	"	14.0%
Flour	1 500 000	"	2.1%
Miscellaneous	0 500 000	"	0.7%

For west-bound cargoes, coal, 18 400 000 tons, constitutes 87% of the tonnage.

Perhaps the valuation is a better criterion of the use of a waterway than the number of tons carried. Of the total valuation of $1 026 045 064, the valuations of various classes of cargo are, as follows:

Lumber	0.69%
Flour, wheat, and other grain	41.93%
Copper	1.65%
Iron ore, pig iron, and manufactured iron and steel	30.51%
Coal, soft and hard	9.98%
Miscellaneous merchandise	15.24%

It should be noted that the valuation of all freight carried, outside of iron ore, is $720 356 808.

The freight rates for various commodities in 1923 were as follows, reduced to cost per ton, including loading and unloading except in the case of coal:

Lumber	$2.75
Flour	3.50

Wheat	$1.27
Other grains	1.52
Copper	5.00
Iron ore	0.83
Pig iron	2.85
Manufactured iron and steel	2.50
Coal	0.45*
Salt	1.25
Oil	1.25
Stone	0.50
General merchandise	5.00

These freight charges bear some relation to the value of the commodity itself. Lumber has a value of $18.65 per ton; flour, $62.50; wheat, $35.30; grain other than wheat, $29.60; copper, $281.00; iron ore, $5.15; pig iron, $30.85; manufactured iron and steel, $85.00; coal, soft, $5.00; coal, hard, $11.20; salt, $8.90; oil, $41.00; stone, $1.10; and general merchandise, $225.00.

It is obvious also that the space occupied by freight of a given class, as well as the difficulties of loading and unloading, enter into the freight rates charged. Taking 100 cu. ft. as the measure of the registered ton of vessel capacity, 1 ton of wheat occupies 37⅓ cu. ft. of space; 1 ton of lumber occupies 40 cu. ft.; and 1 ton of general merchandise is understood to occupy 100 cu. ft., which figure, however, has wide variations. The freight passing the Soo Canal in 1923 shows 1.43 tons of freight carried for each registered ton of vessel capacity. This means that the average ton occupied 70 cu. ft. of space.

These statistical figures are tedious and need to be read only by those who wish to analyze the water-transportation problem in its bearing on the land-locked commerce of the United States and the possibilities of emergence of this commerce into the salt seas. The big problem aside from carrying capacity, appears to be that of increasing the load factor of vessels of the Great Lakes by the stimulation of up-bound cargoes and by 12-month operation. Coal shows a marked upward volume of tonnage, the coal carried in 1900 being less than 4 500 000 tons, while that carried in 1923 was more than 18 400 000 tons. Grain cargoes are also increasing, showing for a period about 1900, 73 000 000 bushels and for 1923, 370 000 000 bushels. Lumber, on the other hand, shows a declining tonnage. The growth in the transportation of general merchandise is very small.

Forecast

For the coming decades it is probable that the shipments of high-grade, commercial, hematite iron ore will decline. This decline will be replaced, in part at least, by the shipment of ores of lower grade which will be concentrated before shipment by the removal of some non-metallic elements. The

* Does not include loading and unloading.

shipment of coal will increase largely with each decade, tending more nearly to equalize up-bound and down-bound cargoes. The greater carrying capacity of vessels which will come with deeper drafts, the near-balance of up-bound and down-bound cargoes, and the final utilization of some vessels now limited to an 8-month year for ocean service during the four ice months, will all tend to the further reduction of freight rates.

It is interesting to note the economic value of the present commerce of the Great Lakes, excluding iron ore and coal. Aside from these commodities, the 1923 shipments on the Great Lakes aggregated, for the Soo Locks, 13 627 404 tons, and the freight paid was $23 398 840. These shipments, assuming an average travel of 800 miles, represent a ton-mile rate of 2.14 mills. A normal rail rate per ton-mile is 11 mills. The saving by this part of the transportation on the Great Lakes over rail on these commodities alone represents an excess of more than $90 000 000 for 1923.

Assuming the elimination of iron ore alone, and considering coal and all other commodities, and taking a rate of $1.00 per ton on up-bound coal, the aggregate carried through the Soo Locks in 1923 is 32 022 715 tons and the freight charged would be $41 694 150. The average rate per ton-mile then becomes 1.63 mills. This is less than one-sixth the normal rail rate. It should be understood that if it were not for the cheap freight rates on the Great Lakes, much of the iron ore of Minnesota might lie unused in the ground, the ores coming from other sources; and coal might come by rail from Illinois and Kentucky instead of from the Pennsylvania mines. About 80% of the iron used in the United States comes from the mines tributary to Lake Superior. It is probable that ultimately iron ore will be exported from Minnesota.

The Soo to Port Huron

The 600-ft. freighter coming down Lake Superior from Duluth and reaching the Soo Locks 36 hours later has spent 1½ hours in passing through the locks and canals, including the examination of statistics. The descent in the locks is about 20 ft. under normal conditions. Leaving Sault Ste. Marie, down-bound, the vessel course follows for 2 miles the partly improved river with a width of ¾ mile, then for a distance of 5 miles in a dredge cut through Little Rapids into Hay Lake. This lake is about a mile wide and, for 4 miles, has depths requiring little or no dredging to secure 25-ft. drafts. The channel splits about midway of Hay Lake, down-bound vessels taking the West Neebish Channel and up-bound the Middle Neebish Channel for a distance of 14 miles, when the course leads into Mud Lake and the estuary of St. Marys River—upward of 2 miles wide. From here to Point Detour is 25 miles of clear sailing in a channel the greater part of which is a mile or more in width. At Point Detour, vessel courses radiate toward Lake Michigan and Lake Huron ports. Vessels bound for Chicago, work to the westerly in Lake Huron and through the Straits of Mackinaw to the foot of Lake Michigan, 423 miles away. The down-bound 600-ft. freighter passes down Lake Huron with depths as great as 400 ft., reaching Port Huron, Mich., at the head

of the St. Clair River, in 216 miles of travel and 20 hours after leaving Point Detour. This is 62 hours from Duluth.

Suggested Works

At the foot of Lake Huron—at the head of the St. Clair River—the writer suggests regulating works to maintain lake levels and create storage. Such regulation means that the elevation of Lake Huron will be normally 1 to 2 ft. or more higher than the water in the head-of-the-river pool opposite Fort Gratiot Lighthouse. The elevation of the water at Fort Gratiot primarily determines the outflow through the St. Clair River; and conversely the surface elevation or stage of Lakes Michigan-Huron is determined by the abundance or meagerness of the water supplied to these lakes. It is purposed to curtail the natural prodigal outflow from the Michigan-Huron Reservoir in times of opulent water supply and during the winter so as to store the water against periods of meager water supply; and it is proposed to release reservoired water, when Lakes Erie and Ontario need navigational uplift and when hydro-electric plants in the Niagara and St. Lawrence Rivers require volume maintenance. Regulating works above the head of the St. Clair River will co-ordinately require navigational passes to permit up-bound vessels to ascend from the head-of-the-river pool level to the lake level. (See Fig. 5.) The 600-footer, down bound, will enter the 600-ft. wide pass and traverse without delay the three expansion basins proposed. Up-bound vessels will encounter no inconvenience or delay. No ship locks will be required.

Port Huron to Detroit

At present, the down-bound 600-ft. freighter passes directly into the gorge of the St. Clair River at Port Huron, 800 ft. wide, 50 ft. deep and flowing with a velocity of 5 miles per hour. Just below this rapid descent of the upper river is a mile of duplicate channels. For the greater part of 27 miles the river is more than ¼ mile wide, with water varying from 25 to 50 ft. deep. The current, except at Port Huron, is leisurely. Just below Algonac, Mich., 28 miles below Port Huron, the river enters a delta formation radiating into a series of channels somewhat similar to those at the mouth of the Mississippi. The material to be excavated in artificial channelization is alluvial. For a distance of 11 miles, the channel is 500 to 1 000 ft. in width, with some curvature. Ultimate deep channels will involve rectification as well as deepening of some portions of this reach. The 600-ft. freighter passes into Lake St. Clair through the St. Clair Flats Canal with duplicate channels. Lake St. Clair is practically an enlargement of the rivers between the foot of Lake Huron and the head of Lake Erie. The bottom is largely of fine sand, with some clay. The vessel track across Lake St. Clair is 15 miles long and has a width of 600 to 800 ft., much of which has been artificially deepened by the stirring up of the bottom by the propeller wheels of the many passing vessels. Although the natural depth of at least 10 miles of Lake St. Clair was about 21 ft., the channel dredged by vessel travel has a depth of about 24 ft. This deepening of channels by propellers means simply the roiling of

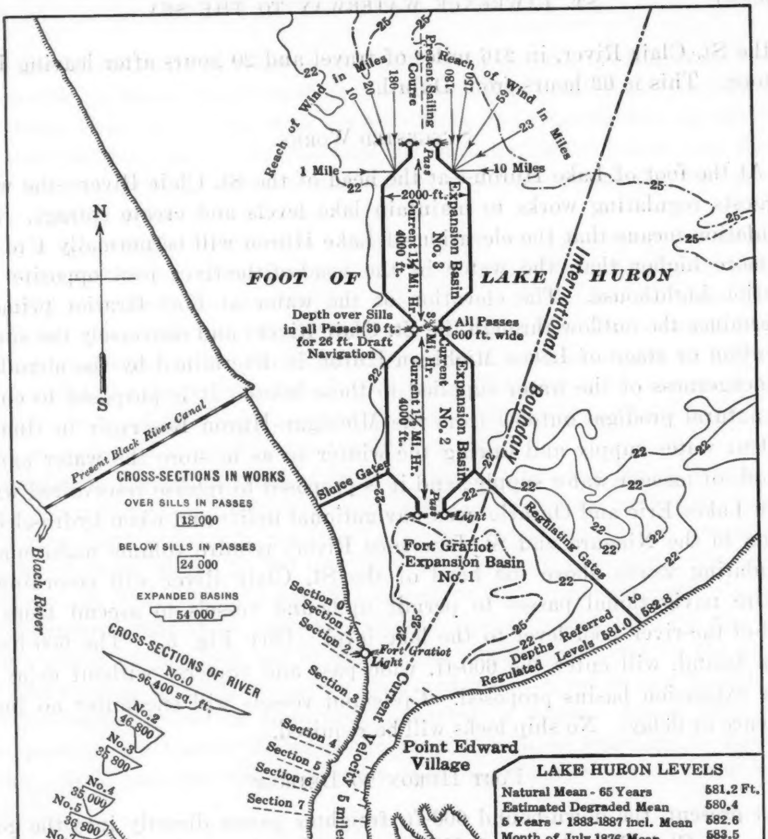

FOOT OF LAKE HURON
Reach of Wind in Miles
Reach of Wind in Miles
Present Sailing Course
1 Mile
10 Miles
2000 ft.
Expansion Basin No. 3
Current 1½ Mi. Hr.
4000 ft.
Depth over Sills in all Passes 30 ft. for 26 ft. Draft Navigation
All Passes 600 ft. wide
Expansion Basin No. 2
Current 1½ Mi. Hr.
Sluice Gates
Light
Fort Gratiot Expansion Basin No. 1
Regulating Gates
International Boundary
Depths Referred to Regulated Levels 581.0 582.8
Present Black River Canal
Black River
N
S
CROSS-SECTIONS IN WORKS
OVER SILLS IN PASSES
18 000
BELOW SILLS IN PASSES
24 000
EXPANDED BASINS
54 000
CROSS-SECTIONS OF RIVER
No. 0 96 400 sq. ft.
No. 2 46 800
No. 3 25 800
No. 4 35 000
No. 5 36 800
No. 7 41 900
No. 8 44 700
No. 9 41 500
No. 10 43 000
No. 11 56 900
No. 12 59 500
No. 13 64-900 sq. ft.
No. 14 60-300
No. 15 56 500
Section 0
Section 1
Section 2
Fort Gratiot Light
Section 3
Section 4
Section 5
Section 6
Section 7
Section 8
Section 9
Section 10
Section 11
Section 12
Section 13
Section 14
Section 15
Current Velocity 5 miles an hour
Point Edward Village
SARNIA BAY
ST. CLAIR RIVER
BLACK RIVER
International Boundary
Sarnia, Ontario, Canada
Port Huron, Michigan
VERTICAL SCALE OF SECTIONS
0
70
LAKE HURON LEVELS
Natural Mean - 65 Years 581.2 Ft.
Estimated Degraded Mean 580.4
5 Years - 1883-1887 incl. Mean 582.6
Month of July 1876 Mean 583.5
Jan., Feb., Mar., Apr., 1925 Mean 578.4
Jan., Feb., Mar., Apr., 1896 Mean 579.1
Proposed Regulated Mean 582.8
In Head-of-River Pool, for Discharge of 200 000 C. F. S. Surface Level 581.0
PROPOSED NAVIGABLE PASSES EXPANSION BASINS AND REGULATING WORKS AT HEAD OF ST. CLAIR RIVER
Scale in Feet
1000 0 1000 2000 3000 4000 5000
ST. CLAIR
TUNNEL — CANADIAN NATIONAL RAILWAYS
MAP COMPILED FROM U. S. LAKE SURVEY DATA

FIG. 5.

the water, with a portion of the material thrown into suspension depositing outside the channel. This condition is possibly pertinent in the consideration of the maintenance of Lake St. Louis, Lake St. Francis, and Lake St. Peter in the St. Lawrence River where the alluvial material is somewhat similar. Lake St. Clair is normally 5.5 ft. below the level of Lake Huron and 3.1 ft. above the level of Lake Erie.

Detroit

The center of the Port of Detroit is 6.5 miles below the head of the Detroit River and the foot of Lake St. Clair. The river at this point is ½ mile wide, with depths from 30 to 45 ft. and a current of perhaps 2 miles per hour. The harbor front extends practically from the head of the river down for a distance of 13 miles. On the north bank of the river is the Canadian City of Windsor. Additional harbor frontage is given by the improvement of the Rouge River—5 miles down from the center of Detroit—where Henry Ford has extensive works. The Port of Detroit is 724 miles from Duluth, involving 66 hours of vessel travel. The name, Detroit, means a strait, and this river, 27 miles in length, connecting Lake St. Clair with Lake Erie, varies in elevation with Lake Erie heights which are reflected by back-water into Lake St. Clair.

Detroit to Buffalo

Southward, for a distance of 14 miles below the center of the Port of Detroit, the river is capacious, requiring little or no dredging to develop it. Then the river shallows, showing estuary conditions. A little lower down, the channel splits, down-bound vessels following the Livingstone Cut, and up-bound vessels taking the old improved Canadian Channel. This reach of the river requiring improvement has a length to Bar Point, Ont., of 6.5 miles; and for 3 miles more through the delta formation of Lake Erie additional dredging will be required.

From Bar Point to the Port of Buffalo is 240 miles of clear sailing in open Lake Erie. This lake is the shallowest of all the Great Lakes, having depths for the most part of 100 ft. or less. The west end of the lake shows depths of 25 to 35 ft. for a distance of 30 miles. On account of the tendency of Lake Erie to tilt up in times of severe storms, channels have been dredged to greater depths than in the other more stable lakes. It will be shown subsequently that the betterment of Erie levels is involved in this storm-tilting of the lake.

The Port of Buffalo is 985 miles and 90 hours travel from Duluth. The entrance to the Welland Canal at Port Colborne, Ont., is 10 miles less. These ports represent the present eastern limit of deep-draft navigation.

The waters of Lake Erie discharge through the Niagara River with its head at Buffalo. The Board of Engineers for Rivers and Harbors of the Corps of Engineers, U. S. Army, recommended in 1920, immediate construction of regulating works at the head of the Niagara River to maintain Lake Erie levels. The writer will discuss works at this point in connection with the control of all the outflow rivers of the Great Lakes for the dual purposes of the maintenance of the best navigable depths and at the same time the creation of storage for water power in the Niagara and St. Lawrence Rivers.

Outlets to the Sea

The navigable waterway of the Great Lakes with its deep penetration of the continent (see Plate II), and its large commerce cannot remain always a land-locked interior basin without adequate connection with the salt seas. For a century the desirability of this connection has been obvious and considerable steps toward its achievement have already been made. The Erie Canal was begun in George Washington's day and about the same time the Canadians were building waterways entering the Great Lakes System by the way of the St. Lawrence River and Lake Ontario and then over the Niagara Escarpment through the Welland Canal to Lake Erie. In the summer of 1924, the centenary of the beginnings of the Welland Canal was celebrated. The present Welland Canal with 14-ft. drafts and lock chambers, 270 ft. long by 45 ft. wide, descends from Lake Erie to Lake Ontario with a definite, although small, volume of commerce, perhaps showing a maximum of 3 500 000 tons a year; and 2 000-ton vessels passing through the Welland Canal go on down to Montreal through the Laurentian Canals which have drafts and locks much the same as those of the Welland. The New York State Barge Canal leaving the Niagara River at Tonawanda, N. Y., a few miles below Buffalo, is an escape to the Hudson River, and thence to salt water in the Port of New York. The Illinois Waterway at present under construction, will pass through the Drainage Canal at Chicago to the Des Plaines and Illinois Rivers with 9-ft. drafts and thence by the Mississippi River to the Gulf of Mexico. The use of any one of these three canals involves either the utilization of light-draft vessels on the Great Lakes or transshipment from the deep-draft lake carriers. Neither of these alternatives makes for the most economic through transportation.

The Welland Ship Canal

Before discussing the various deep waterways to the sea, proposed a quarter of a century ago, it may be desirable to note the near completion of the New Welland Ship Canal, assume that this is the year 1930, and continue toward Montreal on the 600-ft. freighter down-bound from Duluth.

The Welland Canal entrance is at Port Colborne. It is protected from storms, and mooring space is provided by an extension of breakwaters and piers for a distance of more than a mile out into Lake Erie. The entrance to this harbor and the harbor itself are dredged to a depth of 30 ft. below ordinary Lake Erie level. The object of this dredging is to provide for the tilting of Lake Erie in storm-time, so that when the water level drops 5 ft., vessels drawing 25 ft. of water may still enter the harbor. This deep dredging is continued to the Guard Lock, 2 miles from the harbor entrance, where vessels will descend normally about 5 ft. The inner summit canal has a depth of 25 ft., a bottom width of 200 ft., and slopes of various inclinations—the normal water surface width being 305 ft. The canal has a cross-sectional area of 6 000 sq. ft. and is capable of passing, without obstructive currents, about 10 000 cu. ft. per sec. The water-power capacity of this canal will be touched upon later in discussing Niagara River power.

Lift Lock No. 7 is 20 miles from the Erie Harbor entrance. The descent in this lock is, as in the remaining six locks, 46.5 ft.; and the chamber in all the locks is 800 ft. long by 80 ft. wide, with 30 ft. depth capacity. Another ½ mile brings the 600-ft. freighter to tandem Locks Nos. 6, 5, and 4, which are twin locks, as well. The twinning of these locks is necessitated by water supply conditions, all other locks being single-chamber locks. After 5 miles of further travel, including the passing of Locks Nos. 3, 2, and 1, the freighter has descended 326 ft. to Lake Ontario level, which is normally about 246 ft. above mean tide at New York; then 9 600 ft. through a broad canal and an inner harbor—over 1 mile long and 800 ft. wide, with adequate mooring space—to the 400-ft. entrance into Lake Ontario.

The work done on the Welland Canal in its various elements represents the highest engineering achievement. As already stated, it will be completed about 1930 and will cost upward of $100 000 000. The original undertaking was pre-war and the ultimate expenditure of so much money as the final cost was not anticipated. Whether or not the St. Lawrence River itself is canalized for sea-going vessels, this adequate link over the Niagara Escarpment brings Lake Ontario into the deep-water basin of the Great Lakes System leading to Buffalo, Detroit, Chicago, Port Arthur, and Duluth. The estimated time consumed in passing through this 27 miles of waterway, including harbors at each end, is about 12 hours.

Port Weller to St. Lawrence

From Port Weller, the Lake Ontario exit from the Welland Canal, the course is straightaway toward Tibbetts Point at the head of the St. Lawrence River, 156 miles away. Northerly from Port Weller is the Canadian City of Toronto, 29 miles away; and the City of Hamilton is about the same distance to the westward. Charlotte, the lake port of the City of Rochester, N. Y., is 92 miles to the eastward, while Oswego, N. Y., the Lake Ontario entrance to the New York State Barge Canal, is 141 miles to the eastward of Port Weller. Lake Ontario has depths exceeding 600 ft.

Upper St. Lawrence

The 600-ft. freighter passes the Tibbetts Point Light-House and enters the St. Lawrence River, 14 hours after leaving Port Weller—1 157 miles and 115 hours from Duluth. At its head the river is 10 miles across from the New York shore to the Ontario mainland, with the waterway split by islands occupying 6 miles of the width. The formation here is igneous with characteristic pinnacle rocks forming submerged obstructions and creating the Thousand Islands. This condition extends down stream 51 miles to Morristown, N. Y. It should be understood that the description of channels relates to the usual vessel track, crossing back and forth in some places over the International Boundary. In this 51-mile reach natural channel depths range from 30 to 180 ft., and the deep vessel right of way is 1 000 ft. or more in width—except that a 7-mile stretch from Thousand Islands Park to Alexandria Bay and a

3-mile stretch, beginning 4 miles above Brockville, Ont. (which is a mile above Morristown), have widths ranging from 300 to 600 ft.—but with very deep water. (See Fig. 6.) Some cleaning up is indicated in this 10 miles of right of way.

Below Morristown, the river flows 50 ft. deep and ¾ mile or more wide for a distance of 11 miles to Prescott, Ont., and Ogdensburg, N. Y.—at the mouth

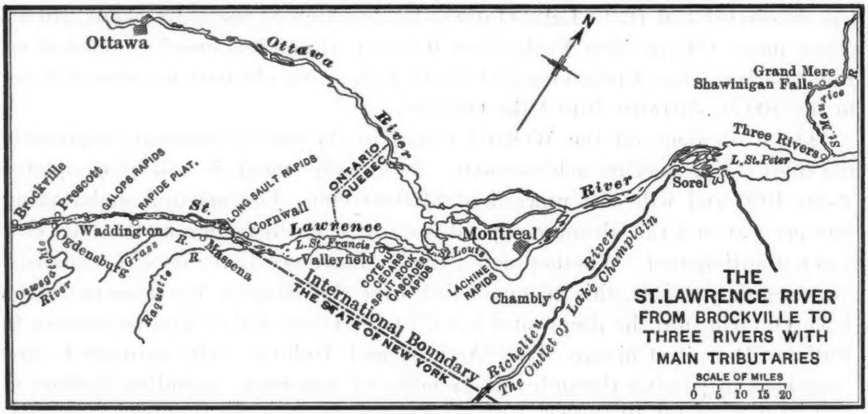


Fig. 6.

of the Oswegatchie River. These cities are 62 miles down the St. Lawrence—1 219 miles and 120 hours, or 5 days—from Duluth.

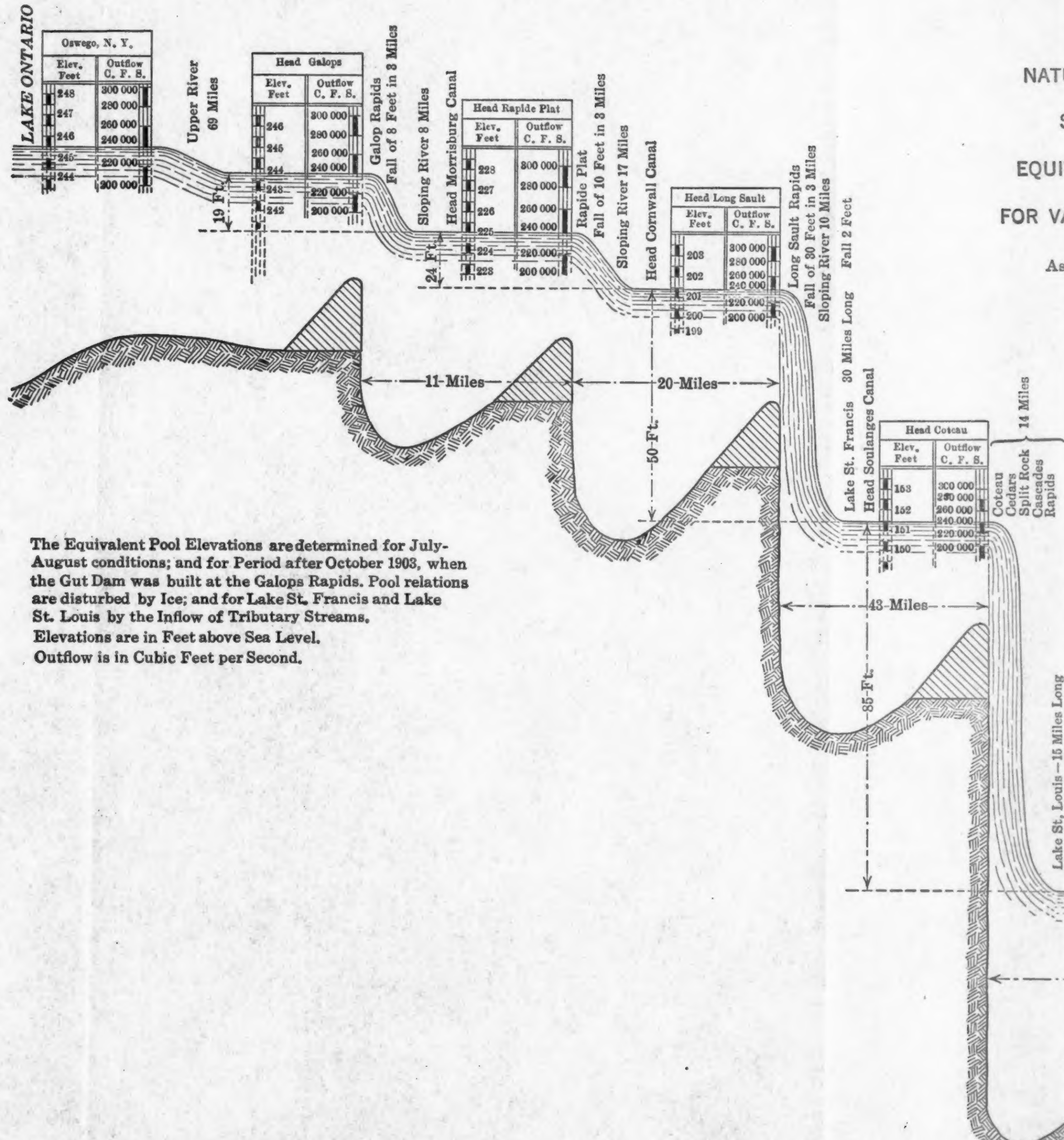
The deep wide river extends 3.5 miles down stream from Prescott to the entrance of the Canadian Cut known as the North Channel—with the head of the Galops Rapids 3 miles farther down stream. On the New York side, a deep channel extends 6 miles to the head of the American Galops Rapids. A limestone ledge, overlain in part with glacial drift, forms the natural weir which creates Lake Ontario. (See Plate IV.) The upper 69 miles of the St. Lawrence River is hydraulically an arm of the lake—with an open-season descent of only 1 ft. from lake level to river level at Ogdensburg.

As the St. Lawrence River takes a general northeasterly course, it is well to note the fact that the latitude of Ogdensburg is 44° 42′, and the latitude of Liverpool is 53° 24′—so that Liverpool is still 600 miles to the north of Ogdensburg; and when an ocean liner finally passes from the Gulf of St. Lawrence into the open Atlantic, it is still 100 miles to the south of Liverpool, and about 200 miles to the north of Havre.

International Rapids Section

The zero point for the Rapids Section of the St. Lawrence Waterway begins about 4 miles below Ogdensburg. It is desirable to note the characteristics of the St. Lawrence River and the flanking canals used by lake-going vessels with a draft of 14 ft. and carrying perhaps 2 000 tons of freight.

Down-bound vessels pass through the North Channel, an artificial rock cut, for a distance of 1½ miles, then another 1½ miles to the entrance of the Galops



The Equivalent Pool Elevations are determined for July-August conditions; and for Period after October 1903, when the Gut Dam was built at the Galops Rapids. Pool relations are disturbed by Ice; and for Lake St. Francis and Lake St. Louis by the Inflow of Tributary Streams.
Elevations are in Feet above Sea Level.
Outflow is in Cubic Feet per Second.

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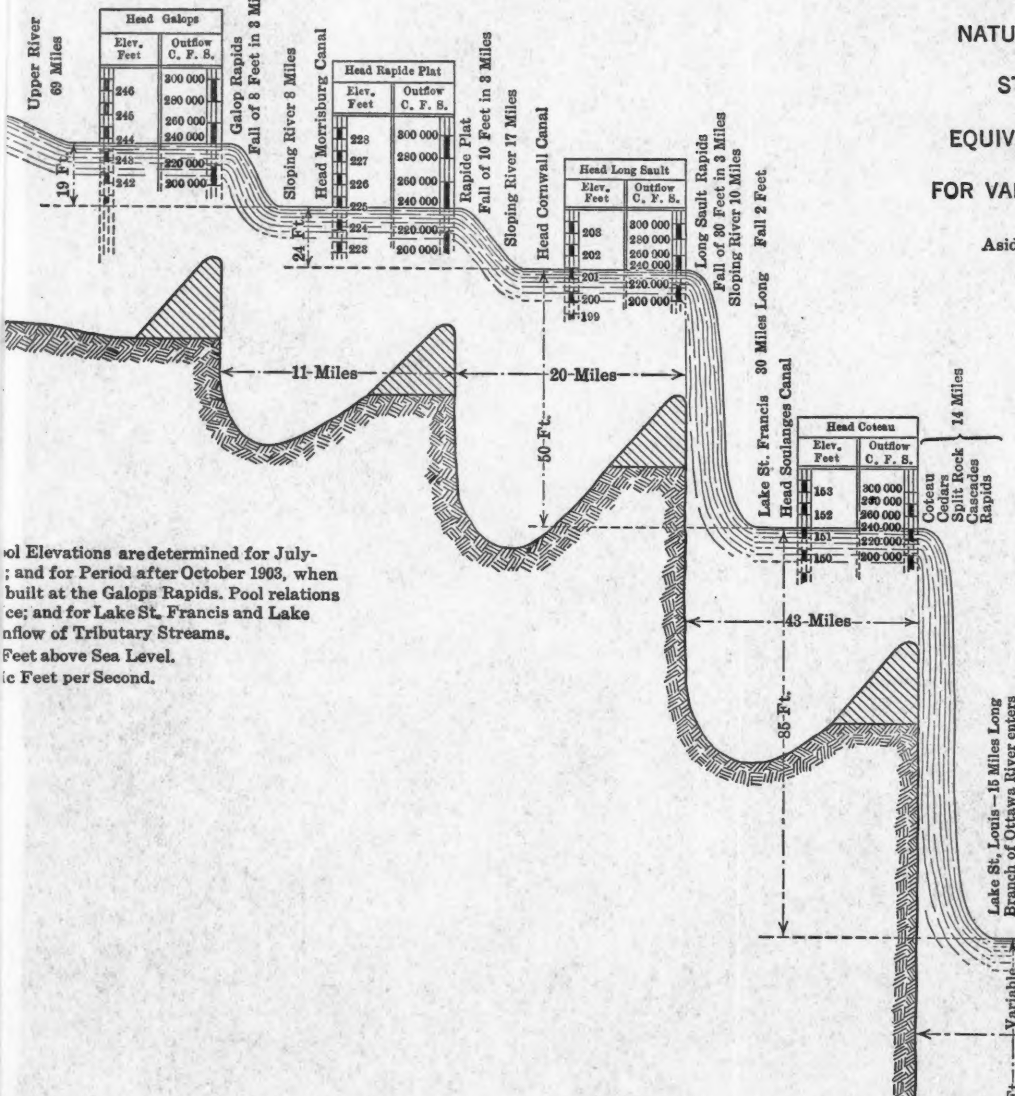

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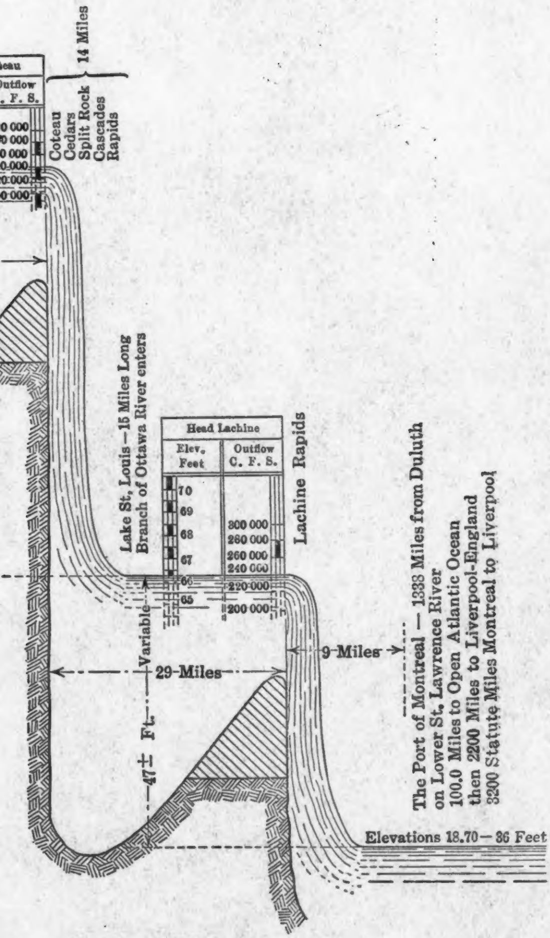
PLATE IV.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1925
SHENEHON ON
THE ST. LAWRENCE WATERWAY
TO THE SEA

NATURAL POOLS AND WEIRS OF THE ST. LAWRENCE RIVER WITH EQUIVALENT WATER SURFACE ELEVATIONS FOR VARIOUS VOLUMES OF FLOW

Aside from Gage-Volume Boards this Plate is not to scale



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Canal at the head of the Galops Rapids. Some down-bound vessels chute the rapids following an improved channel through the rock barrier. Up-bound vessels take the Galops Canal. The descent in the Galops Rapids is 8 ft. in a distance of 3 miles. The canal extends for a distance of 7 miles flanking the swift, curving river, descending 14 ft. in the terminal lock.

At the Town of Iroquois, Ont., vessels enter the river again, following the deep channel in a swift current for a distance of 4 miles to the entrance of the Morrisburg Canal at the head of Rapide Plat. This canal has a length of 4 miles, making a descent of 12 ft. in the locks at Morrisburg, Ont. Down-bound freighters sometimes chute these rapids. The visible bottom in the upper portion of these rapids is of red sandstone and it is in this vicinity that the engineers of the International Joint Commission have recommended regulating works, for the maintenance of Upper St. Lawrence River and Lake Ontario levels, the creation of storage, and the manual control of the water supplied to the river.

After passing through 10 miles of swift-flowing river with depths of 20 to 40 ft. or more, the narrow channel and swift current at Croil's Island is reached. Down-bound vessels follow the river through this swift water, descending 3 ft. in less than a mile, while up-bound vessels take the flanking Farran's Point Canal. At the head of Croil's Island, the channel splits, leading to the right toward the South Passage of the Long Sault Rapids and the entrance 4 miles down from the splitting point to the Massena Power Canal. Vessels passing out of the Farran's Point Lock take the capacious, deep, north channel for a distance of 4.5 miles to the entrance of the Cornwall Canal and the head of the Long Sault Rapids. Only a few passenger steamers chute these and the other magnificent rapids between this point and Montreal. The Cornwall Canal makes a descent of 50 ft. in 11.5 miles. The Long Sault Rapids make the major descent in a main-channel distance of 3 miles. The descent to Lake St. Francis, with a volume flow of 230 000 cu. ft. per sec., creates the dominant water power of the International Section of the St. Lawrence River.

The Canadian Rapids Section

About 1.5 miles below the lowermost lock at Cornwall, Ont., the International Boundary Line leaves the St. Lawrence River and proceeds overland, following eastward on the 45° parallel of latitude.

From this point, 46.5 miles below the zero point of the International Rapids Section, the Canadian National Section begins. This is at the head of Lake St. Francis and is 50.5 miles down stream from Ogdensburg and Prescott and 1 270 miles from Duluth. From the level of the St. Lawrence River at the head of the Galops Rapids, the descent to the head of Lake St. Francis is 91 ft.; and Lake St. Francis is still 152 ft. above sea level.

The conditions in the Canadian National Section are exceedingly simple: First, Lake St. Francis—30 miles long, with a width of 1 mile to 5 miles, and water abundantly deep for 25-ft. navigation—with the exception of about 3 miles which need dredging or cleaning up. The total area of the lake is about 90 sq. miles which will give excellent forebay capacity for peak-load operation

of hydro-electric plants in the tandem rapids at its foot. These outflow rapids in series, beginning at the entrance of the Soulange Canal at Coteau Landing, are the Coteau, the Cedars, the Split-Rock, and the Cascades—the full series making a descent in 14 miles of 84 ft. to Lake St. Louis. Lake St. Louis is another capacious, deep lake, 15 miles long, with widths ranging from 1 mile to 6 miles. It has a forebay value of over 50 sq. miles to stabilize varying volumes of flow in the hydro-electric plants of the future in the outflowing Lachine Rapids. The Lachine Rapids, beginning at the head of the Lachine Canal, make a normal descent of 45 ft. in 9 miles to the Lower St. Lawrence at the ocean port of Montreal—with the Atlantic 1 000 miles away down a magnificent, still-flowing river—with 30-ft. drafts on the trans-Atlantic liners, an accomplished fact.

The Port of Montreal

The Port of Montreal has been created by the expenditure of nearly \$40 000 000. It has deep-draft berths to accommodate over 100 large modern ocean steamships simultaneously, with over 8 miles of wharves and piers capable of docking vessels drawing over 25 ft., and over 5 miles of these will receive vessels drawing 30 ft. and over; 60 miles of railway tracks to serve these wharves; 35 of these steamship berths are modern concrete wharves built in the past few years; 4 large modern fire-proof elevators with a capacity of 12 000 000 bushels, having conveyor systems to 26 steamship berths at which 19 vessels can be loaded with grain at one time; 24 permanent fire-proof transit sheds; modern cold storage warehouse; complete and valuable construction and repair plants; and near-by shipbuilding plants, including a dry dock. These facts are taken from the 1923 report of the Montreal Harbor Commission.

Having in mind the large financial investment in creating the modern facilities of a great terminal port, where vessels discharge their import cargoes for transshipment to smaller lake vessels or to rail carriers, and where large quantities of freight for export are received from the Great Lakes either by the small canal type of vessel or by rail, it may be readily appreciated why Montreal may view apprehensively the creation of a deep-draft waterway interfering with this great and profitable business of transshipment. The situation here is somewhat the same as the situation in Buffalo, the present down-bound Lake Erie terminal of the Great Lakes, where cargoes are transshipped to barges of the New York State Canal or to railway carriers.

Since the waterway to the sea between Duluth and Montreal contemplates through shipment without stopping at Montreal for other than supplies, no further detailed statement of the harbor at Montreal is desirable in this paper.

However, as Montreal is the eastern limit of the contemplated improvement, the conditions of navigation and its strategic position with respect to the seaports of the world will be reviewed, before touching on other avenues to the sea, and before discussing the proposed constructions for navigation and water power in the Rapids Sections of the St. Lawrence River for a distance of 115 miles up stream from this port.

St. Lawrence Ship Channel

Below Montreal, dredged channels, 450 ft. wide, with greater widths on curves, and with depths ranging from 30 to 35 ft., are encountered for a distance of 150 miles to Quebec and for a further distance of 175 miles to Father Point, Que. Below this are few dredged areas. At Three Rivers, Que., 80 miles below Montreal, tidal movements are discernible. To April, 1921, the Dominion Government had expended about $24 000 000 on the St. Lawrence Ship Channel below Montreal. The season of navigation begins in late April and ends in early December, having a normal length of 224 days, or 61% of the year. The hazards of navigation are not large and insurance rates will not be prohibitive.

Montreal's Strategic Position

The Port of Montreal is 370 miles nearer Liverpool than the Port of New York. The distance to Gibraltar, the gateway to Mediterranean ports and to the Suez Canal, is approximately the same for the two ports. From the foot of Lake Erie to Liverpool by way of New York is 443 miles by rail and 3 576 miles by sea, a total of 4 019 miles; while the all-water route through the St. Lawrence is 350 miles shorter. The loss of time in passing through the Welland Canal—about 9 hours more than in open channel running—and the loss of time in the St. Lawrence River Section—perhaps 9 hours more—are probably less than the time lost in transshipping cargoes. The handicap of any water route in northern latitudes is the embargo of the ice season of 140 days.

Montreal Commerce

The 1923 commerce of the Port of Montreal amounted to 6 372 400 tons, the exports being a little less than twice the imports. The imports, however, were mostly from the United States and consist largely of coal and petroleum. The dominant export cargo is wheat and other grains, showing a total volume of 3 278 100 tons out of the total export volume of 4 152 900 tons. Other large exports are automobiles and parts, dairy products, flour—346 100 tons—fruit, and meats. Of the exports of wheat and other grains amounting to 120 108 000 bushels in 1923, nearly 75 000 000 bushels reached Montreal by boat. During the same year, the Port of Buffalo received by lake 182 000 000 bushels, of which 114 000 000 bushels were Canadian grain. It is interesting to note the fact that 294 000 000 bushels of grain were shipped from the Canadian Lake Superior Ports of Fort William and Port Arthur between September 1, 1922, and September 1, 1923. A part of this grain was moved by rail. It is also interesting to note that the Duluth-Superior Port shipped in 1923 about 70 000 000 bushels of grain and that the volume of grain passing through the locks at Sault Ste. Marie in 1923 was about 370 000 000. During 1923 Buffalo shipped by the Erie Canal about 20 000 000 bushels of wheat. The total commerce through the Erie Canal was 1 626 100 tons in 1923.

If the value of exports in the Port of Montreal is taken as $50 per ton, the total valuation appears to be $200 000 000.

Lake-and-Ocean Vessels

It will be assumed that engineers will solve the problem of composite vessels to traverse the fresh-water seas and the salt seas; that the evolution of this vessel will come in the ten years between the definite decision to construct the waterway and its completion; and if not in this decade then in some of the early decades of the century. Such a vessel was designed nearly a quarter of a century ago by an eminent naval architect and shipbuilder of Detroit, Frank E. Kirby, and his conclusions—published in 1900 in the report of the Board of Engineers on Deep Waterways—indicated the cost to be practically 10% more than the cost of a vessel built for exclusively Great Lakes navigation. Any economies in transportation by the opening of this waterway might be cancelled by the time losses, by the cargo damage and losses, and by the labor and overhead costs of transshipment. The scheme of water transportation connecting Liverpool, Continental Atlantic, and Mediterranean ports, must contemplate fundamentally the continuous movement of an unbroken cargo. It should be understood that special types of boats will continue in special service, such as ore and coal-carrying, between Lake Superior and Lake Erie ports. It is of little concern whether the Rapids Section of the St. Lawrence Waterway to the Sea has, in its first decade, a commerce of 5 000 000 tons or 20 000 000 tons. The analysis of possible traffic and the savings to be made by a through route from Duluth or Chicago to Montreal and overseas is interminable and speculative. It will appear that the construction of this waterway is inevitable, that the cost will be largely carried by the co-ordinately developed water power, and that a demonstration of its commercial feasibility is unnecessary.

Other Waterways to the Sea

Turning briefly to deep waterways other than through Montreal to pass from the Great Lakes to the Atlantic, it should be recalled that under an Act of Congress of February, 1895, a Deep Waterways Commission was appointed to make a preliminary inquiry concerning waterways between the ocean and the Great Lakes. It was provided that the commissioners appointed under this Act should serve without compensation. The Commission appointed by the President of the United States consisted of James B. Angell, then President of the University of Michigan, John E. Russell, and Lyman E. Cooley, M. Am. Soc. C. E. This Commission made an investigation from existing sources of information and reported voluminously in 1897.*

This Commission recommended the appointment of a Board of Engineers on Deep Waterways. Congress acted on this recommendation and made appropriations ultimately aggregating nearly $500 000. The Board of Engineers appointed consisted of the late Charles W. Raymond, Major, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., Alfred Noble, Past-President, Am. Soc. C. E., and George Y. Wisner, M. Am. Soc. C. E. This Board, after two years of surveys, examinations, investigations, and studies, reported in

* H. R. Doc. No. 192, 54th Cong., 2d Session.

June, 1900.* The report is probably the most elaborate existing discussion of the elements entering into deep waterways.

Routes

The routes contemplated by the Board of Engineers take into consideration the Great Lakes System from Duluth and Chicago to the foot of Lake Erie, with Lake Erie regulated. The descent over the Niagara Escarpment followed the Niagara River to Lasalle, N. Y., for a distance of 16 miles below Buffalo and re-entered the Lower Niagara River at Lewiston, N. Y., substantially at Lake Ontario level, 25 miles from the regulating works at Buffalo; then proceeded through the deep, spacious Lower Niagara River and Lake Ontario to Oswego 139 miles away. Here the canalization climbed up 133 ft. by lockages to the level of Oneida Lake and followed the Mohawk Valley descending to the Hudson at Troy, N. Y. From Troy to the Battery in New York it follows the established tidal Hudson River route for a distance of 140 miles. This route beginning at Oswego and ending in the Port of New York will be referred to as the Oswego-Mohawk Route.

Another route followed Lake Ontario past Oswego, entered the St. Lawrence River at Tibbetts Point, and proceeded to the Galops Rapids. Here, the route followed through a series of river channels and canals, including four ship locks, to Lake St. Francis at the International Boundary. The route then crossed the Boundary Line and followed for a distance of nearly 30 miles down Lake St. Francis to a point on the south bank of the lake opposite Coteau Landing and near the head of the Coteau Rapids. The route then proceeded through the divide for a distance of 40 miles in Canadian territory and for a further distance of 8 miles in American territory to Kings Bay, Lake Champlain—15 miles north of Plattsburg, N. Y. Lake Champlain is about 50 ft. lower in elevation than Lake St. Francis and the descent is made in five ship locks. The route then proceeded 124 miles through deep water in Lake Champlain and through canalized waterways and entered the upper courses of the Hudson River below Fort Edward, N. Y. It met the Oswego-Mohawk route at Troy and proceeded to New York through the tidal Hudson.

Preferred Routes

The Board of Engineers on Deep Waterways recommended the route by the way of Oswego; but since that time the Dominion Government has undertaken and is completing the construction of the New Welland Ship Canal over the Niagara Escarpment, and the development of the St. Lawrence itself to the foot of Lake St. Francis and beyond to Montreal is recommended. The International Section of the St. Lawrence Waterway to the Sea will be subsidized by water-power, and may be eliminated in any estimate of cost chargeable to navigation.

Duluth, Chicago, Montreal, and New York

The preferred route from the Great Lake System to the Port of New York for deep-draft ocean-going vessels now appears to be the St. Lawrence-Cham-

* H. R. Doc. No. 149, 56th Cong., 2d Session.

plain Route; and the aspirations of the citizens of New York for a deep waterway, entering salt water through the Hudson River portal, appears to be in parallel with the aspirations of those seeking a deep waterway following the St. Lawrence River to the Atlantic. The trunk-line is the same for both routes from Duluth for a distance of nearly 1 300 miles, the dividing point being 40 miles above Montreal.

Comparison of Routes

Having in mind this most remarkable community of interest between the State of New York and other States tributary to the Great Lakes, it is well to compare the Oswego-Mohawk and the St. Lawrence-Champlain routes to the sea. From Oswego to the Battery in New York through Oneida Lake and the Mohawk Valley is 313 miles, involving the passage of 31 ship locks, and requires about 46 hours. Reduced roughly to 1924 prices—assumed to be about 50% more than those of 1900—a waterway with 25-ft. depths in canals, canalized rivers, and lakes, and with ship locks, 740 ft. long, 80 ft. wide, and 30 ft. deep, would cost approximately $287 000 000.

By the St. Lawrence-Champlain Route, from the division point at the foot of Lake St. Francis, the distance to New York is 363 miles, involving a descent through 12 ship locks, and requires 42 hours. The cost, reduced to 1924 prices, would be about $221 000 000.

From Buffalo, the distance to New York by the Oswego-Mohawk route is 477 miles, requiring 64 hours' travel. From Buffalo by the St. Lawrence-Champlain route to New York is 685 miles, requiring 76 hours of travel. The St. Lawrence-Champlain route from Buffalo to New York is 208 miles longer than by the way of the Oswego-Mohawk route; and it takes 12 hours longer to traverse it. On the other hand, it costs $66 000 000 less than the shorter route. It would have the advantage of traversing and serving additional territory in New York and New England; and it would not conflict physically with the permanent use of the Barge Canal. If 1924 prices are taken as 15% more than those of 1900, the Oswego-Mohawk route would now cost $335 000 000 and the St. Lawrence-Champlain route $77 000 000 less.

Other Proposed Routes

No other outlet proposed for deep-draft navigation to the sea needs more than casual mention. The Georgian Bay Canal, planned to depart from Lake Huron through the French River, and to pass over the divide to the Ottawa River and thence to Montreal, appears to be disadvantageous in cost, short season of navigation, and in the fact that it would short-circuit the important traffic cities of the Detroit River, Lake Erie, and Lake Ontario. A railway to Hudson Bay has long been advocated and considerable money spent in surveys and investigations. The short season of navigation—about 150 days—and the transshipping elements, are probably fatal to this route in competition with all-water routes.

The St. Lawrence-Montreal Project

The project contemplated in the report of the International Commission* will be lightly touched upon. The investigations of the engineers were limited to a single year, a period much too short to reach other than tentative conclusions.† A new International Board of Engineers has been appointed to review the project, to report in October, 1925. The personnel of the United States and Canadian Government Engineers reporting to the International Joint Commission in June, 1921, was, as follows: For the United States: W. P. Wooten, Colonel, Corps of Engineers, U. S. Army, M. Am. Soc. C. E., H. G. Roby, M. Am. Soc. C. E., Assistant; for Canada: The late W. A. Bowden, M. C. I. E., D. W. McLachlan, Assoc. M. Am. Soc. C. E., Assistant.

Canadian Section

It appears to the writer that the treatment of the St. Lawrence River for navigation and water power from the foot of Lake St. Francis, with a descent of 83 ft. to Lake St. Louis and from Lake St. Louis with a descent of 45 ft. or more to the river at Montreal, is a matter for the Dominion Government to decide. The recommended project contemplates by-passing in lateral canals the rapids from Lake St. Francis to Lake St. Louis and from Lake St. Louis to the lower river. The two proposed 25-ft. depth canals have lengths of 26½ miles and require three lift locks and two guard locks. These locks would have a usable length of 800 ft., be 80 ft. wide, and 30 ft. deep over the breast-walls. The total estimated cost for these two portions of the Canadian National Section is $92 373 000.

The proposed project contributes nothing toward the ultimate water-power development—stated to be 2 420 000 e. h. p.—in these two great descents. The solution of the problem has been based on the absence of a market capable of absorbing the great volume of visible water power in the St. Lawrence River. An alternate navigation route shown in the plans, follows the river channels through the rapids. This would involve the construction of at least three dams, each 40 ft. or more in height, creating deep slack-water pools, with ship locks passing through each dam to the pool below. From the point of view of navigation this appears sounder than the use of lateral canals, but the building of these dams, in the view of the engineers, should await a receptive market for the electric current produced.

It is the writer's conviction that water power should subsidize navigation. The undertaking is, therefore, a commercial project and the United States does not need to contribute toward the cost. The water power belongs to Canada and ultimately should do its share toward financing the necessary ship locks. The solution of the ice problem of the St. Lawrence River probably means the eventual elimination of all rapids.

It appears from the report of the engineers of the Joint Board, that the alternate navigation route following the river channel, with one or two dams in the Lachine Rapids, would cost $7 000 000 more than the navigation route by 13 miles of lateral canals; and the alternate navigation route through the

* Senate Doc. No. 114, 67th Cong., 2d Session.

† The report of the engineers is printed in Senate Doc. No. 179, 67th Cong., 2d Sess.

rapids from Lake St. Louis to Lake St. Francis, with two dams in the rapids would cost $13 119 000 more than through 13½ miles of lateral canals. The addition of $20 119 000 to a navigation project estimated to cost $92 400 000 should be given careful consideration, provided the slack-water pools created give ampler and faster navigable ways, and provided the total cost of the improvement and its maintenance may be paid eventually by water-power rentals. It is difficult to understand how a project of lateral canals, in no wise aiding water-power development, is logically chargeable to anything else than transportation. If, on the other hand, the best and largest water-power development incidentally creates navigable ways—only the operation and maintenance expenditures and interest on the cost of four ship locks—perhaps $20 000 000—are chargeable to navigation. The operation, maintenance, and depreciation estimated by the Commission for 26½ miles of 25-ft. lateral canals—with all permanent structures built for ultimate 30-ft. drafts—is $750 000 per year. This is 0.8% of the capital cost. For the alternate project of river channel slack-water pools this sum should be less. These comments are based on the proposition that all constructions for the navigable waterway should be steps toward and be chargeable to water-power development. The expenditure of $1 158 000 on dredging in the Canadian National Section of Lake St. Francis needs mention only.

International Section

Division No. 4 of the Commission Report embraces the International Rapids Section, from the boundary line at the head of Lake St. Francis to the deep river about 5 miles up stream from the Galops Rapids, covering a distance of 48 miles with a difference of 91 ft. in elevation. Beginning at the head of the Galops Rapids the descent to the head of Rapide Plat—11 miles down stream—is 19 ft.; from this point to the head of the Long Sault Rapids the descent—in 20 miles—is 24 ft.; and from the head of the Long Sault to the boundary in Lake St. Francis, the descent is 48 ft. in 13 miles. (See Plate IV.) The International Joint Commission project for navigation provides a lateral canal, 6 miles long, with two locks, on the Canadian side of the river, following in part the present alignment of the Cornwall Canal around the Long Sault Rapids. A dam at the head of Barnhart Island, raises by 30 ft. the present level of the pool at the head of the Long Sault Rapids to Elevation 231. This gives a total lift from the head of Lake St. Francis of 78 ft. at normal flow stage; and a total power fall of 76 ft. This slack-waters the river for a distance of 17 miles to the foot of a regulating dam in Rapide Plat. The summer slope in the river in this 17 miles would be about a foot. Here, vessels pass through a ship lock, with a lift of 15 ft. to Elevation 247, which is within 1 ft. of the regulated level of Lake Ontario. The regulating dam slack-waters the river to the zero point at the entrance to the North Channel—drowning out the Galops Rapids. The reformed International Section then consists of 41 miles of open-river running, 7 miles of canals, and 3 ship locks.

Duluth to Liverpool

For the whole Rapids Section from the zero point at the North Channel, 3 miles above the Galops Rapids, to Montreal—a distance of 115 miles—the reformed waterway shows 81 miles of open river running, 33 miles of canals, 7 lift locks, and 2 guard locks. An ocean-going vessel with a normal speed of 11 miles per hour will require 20 hours to traverse the Rapids Section to Montreal. This then shows from Duluth to Montreal 1 338 miles, and 140 hours, or 5 days and 20 hours, in transit. The average speed is about 9½ miles per hour. The Board of Engineers on Deep Waterways in discussing the various waterways to the sea based the time on a vessel traveling in open running 12½ miles an hour—and the Board used a quicker locking schedule than the writer. Using the Board's values the time from Duluth to Montreal would be 124 hours, or 5 days and 4 hours. Adding for Montreal to Liverpool 10 days and 20 hours, shows 16 days in transit; and with a day at each end for layover—loading at Duluth and unloading at Liverpool—shows a total time of 18 days. With a cargo of 10 000 tons, the work done is 45 400 000 ton-miles. It is probable that bulk freight may be carried between Duluth and Liverpool at a rate not exceeding 0.7 mills per ton-mile, or $3.18 per ton. This means for wheat a rate of about 10 cents per bushel. The rate in 1923 through St. Marys Falls Canal was 3.8 cents per bushel from Superior to Erie ports, including loading and unloading.

Exporting Raw Materials

An economic principle enters in considering a project which facilitates the export of raw materials. This principle was enunciated by the millers of both the United States and Canada at the time of the hearings before the International Joint Commission: A country's prosperity lies in converting, in its own domain, raw materials into finished products. Wheat should not be exported until it has been converted into flour. The flour mill employs labor, it employs the makers of machinery, sacks, and barrels. It sends the feedstuffs to the farm for poultry and cattle. Export flour, dairy products, poultry, beef—not wheat; export—not ore—but steel!

It is not improbable that on the completion of a waterway to the sea a special type of vessel will carry flour direct from the Great Lakes ports to overseas ports. The present discrimination of rates on flour, as compared with wheat, seriously handicaps export flour trade. In 1923 the total exports of wheat flour from the United States as reported by the Department of Commerce, amounted to 16 310 000 bbl. Of this, 5 218 000 bbl. passed through the Port of New York. During the same year the export of wheat amounted to 98 523 000 bushels. In 1921 the export of wheat was 280 058 000 and, in 1922, 164 692 000 bushels. The largest portions of the 1921 exportation went to Belgium, France, Germany, Italy, The Netherlands, and England, a considerable portion going to Canada, doubtless for export through the Port of Montreal. The largest portion of flour exports goes to the same countries, with additional exports to China and Japan.

Waterway Desirable

The immediate construction of the final link of the navigable waterway from the foot of Lake St. Francis to Montreal at a cost of about $100 000 000, the United States sharing in this expenditure, will probably be in the interest of economic transportation for overseas trade. The most careful investigation, however, should be made, weighing the desirability of spending all the money in a way helpful to ultimate water power. If a somewhat larger expenditure will count toward the water-power development of a decade later, it may be advisable to make this larger expenditure now. The part of the United States would then be to pay the interest on a portion of the bonds until water-power rentals do so.

Water Power and Navigation

Water-power projects are sometimes suspected of wearing the guise of navigation projects. A navigation project appeals more strongly to the minds of the people in the trade districts tributary to the Great Lakes far from the water power. The radius of economic benefits from water power may be limited to 200 miles, while the economic benefits of navigation, reaching the west through Chicago, Duluth, and Port Arthur, may have a radius of 2 000 miles. Many of the people looking for benefits in transportation find it difficult to understand that a project may be dual—that the development of the navigable waterway and the development of the water power go hand in hand, each assisting the other. If the water-power element of the St. Lawrence Waterway to the Sea is too greatly accentuated, it arouses suspicion in the minds of the people of the agricultural regions of the Middle West. It is best, however, to state the case as it exists.

Water Power

The Government engineers find the total potential energy available in the Rapids Section of the St. Lawrence River to be 4 064 000 e. h. p. This is as follows:

Canadian:	Lachine Rapids	860 000 e. h. p.
	Coteau, Cedars, Split-Rock, Cascades	1 560 000 " " "
	Total	2 420 000 e. h. p.
International:	Long Sault Rapids, initial	1 464 000 e. h. p.
	" " " , later	180 000 " " "
	Total	1 644 000 e. h. p.
	Rapids Section, Grand total	4 064 000 e. h. p.

These estimates of potential power are based on a volume of continuous flow of 210 000 sec-ft.; and the use of an over-all efficiency of 88%, with the performance of new units at Niagara Falls as a precedent for this high

efficiency. Only the development of 1 464 000 e. h. p. in the Long Sault Rapids, amounting to 36% of the whole, is contemplated in the initial project. Of this amount, 58 000 e. h. p. is to care for existing needs of the Massena Plant of the Aluminum Company of America.

The volume of continuous or prime flow, taken as 210 000 sec-ft., assumes the diversion in the Drainage Canal at Chicago as discontinued. The writer believes it is safer to assume that the diversion of 10 000 sec-ft. will continue at Chicago for a number of years. The available continuous or low-water flow, as determined by the Government engineers, may be taken as 200 000 cu. ft. per sec. Perhaps the over-all efficiency of 88% is somewhat optimistic. The writer prefers 84 per cent. With these values the available prime power in the St. Lawrence River becomes 2 190 000 e. h. p. in the Canadian Section and 1 410 000 e. h. p. in the International Section. This latter value assumes a head of 74 ft. for all water used at the Long Sault Rapids.

Heads

The heads estimated by the Government engineers in deriving the values first stated are for the Lachine Rapids, 41 ft.; for the Coteau, Cedars, Split-Rock, and Cascade Rapids, 74 ft.; and, initially, for the Long Sault Rapids, 74 ft. The development at the Long Sault Rapids contemplates the ultimate raising of the head about 8 ft. The limitation on available head comes with the increased slope due to greater frictional resistance of the river running under an ice cover. The following may be regarded as reasonably conservative values for the ultimate winter head:

Canadian:	Lachine Rapids	30 ft.
	Coteau, Cedars, Split-Rock and Cascade Rapids	82 "
	Total	112 ft.
International		88 "
	Total	200 ft.

The reduced slope giving the highest usable head means slower current and less ice trouble during the season of zero weather. The reduction of slopes with deeper channels and slower velocities is also desirable from the point of view of navigation.

Prime Water

The dependable volume of river flow is a most important element in the determination of the power content of the St. Lawrence River. In a state of Nature, monthly mean volumes range from as low as 190 000 sec-ft. to 320 000 sec-ft. The outflow follows the caprice of natural automatic control of the waters of the Great Lakes. Colonel Wooten found, by a careful investigation, that the use of regulating works on rock foundations at Rapide Plat, would increase the dependable flow to 210 000 sec-ft. In a report on the regulation of the St. Lawrence River, in 1919, the writer reached conclusions which do not differ greatly from those of Colonel Wooten, although his analysis had in mind navigation as prior to water power in benefits.

These studies of the possibilities of making uniform and dependable the flow of the St. Lawrence River are illuminating as pointing out the possibilities of converting secondary power into primary power by conserving the water and stabilizing the flow. It appears that the regulation of Lake Ontario, with a pull-down of 3 ft., adds 20 000 cu. ft. per sec. to the prime outflow of the St. Lawrence River. It is obvious that the full utilization of all the great natural reservoirs of Lakes Superior, Michigan-Huron, and Erie, added to the Lake Ontario reservoir, will yield the largest volume of prime flow. Although Lake Ontario is a considerable reservoir in itself, with an area of 7 240 sq. miles, the combined area of the three other great reservoirs mentioned is more than twelve times as large. The storage reservoirs and forebays of the Great Lakes will be discussed later in some considerable detail. The available dependable prime flow of the St. Lawrence River with full Great Lakes regulation is given in Table 2.

The years included in this hydrological analysis (Table 2) are more precise than earlier years, in the necessary lake elevations. The first year is 1891 and the last 1924, 34 years in all. The gauge readings shown in Column (2) of Table 2 are of January 1 of the year concerned, determined by taking a mean of the mean for the month of December of the year before and of January of the year concerned. They mark the fluctuations of the lakes from the first day of each year to the first day of the following year. They may be regarded as readings on gauges having the following zeros for the several lakes: Lake Superior, 600.5; Lakes Michigan-Huron, 580.0; Lake Erie, 571.0; and Lake Ontario, 245.0. These may not represent the best final values for these gauge zeros, as subsequent analyses made more in detail may indicate slight changes. The values used will give an additional depth of approximately 2 ft. over the improvement planes for navigation in the lakes, except for Superior, where the benefit may be little over 1 ft.

The approximate mean elevations of the various lakes during the important months of navigation, May to November, inclusive, may be 0.7 ft., the cresting elevation, 1 ft., and the annual mean, 0.5 ft., above the mean elevation for January 1 of each year as shown in Column (2) of Table 2.

The water supply shown in Column (3) of Table 2 is the yield or outflow in the St. Lawrence River after converting the storage changes in the Great Lakes System into second-feet. It is the yield which would have occurred in the absence of any rise or fall in the lake surfaces. It should be understood in interpreting the water supply that the volume of diversion at Chicago, since the opening of the Drainage Canal in January, 1900, has been credited as outflowing in the St. Lawrence River instead of at Chicago. Account was taken of the lessened outflow of the St. Lawrence River due to natural ice conditions.

The dependable flow with limited rise and fall in the various lakes is 230 000 cu. ft. per sec. It was found that the excess flow, with a mean value of 7 100 sec-ft., is undependable. For periods of from 1 to 9 years no excess water may be available. Storage changes are shown in Columns (6) and (7) of Table 2.

TABLE 2.—St. Lawrence River Outflow with Great Lakes Regulation.

Year.	Lake surface, gauge readings, in feet.	Water supply, in second-feet.	Outflow, in second-feet.		Storage changes, in second-feet.	
			Prime.	Excess.	Saved.	Spent.
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1891	2.00	187 000	230 000	......	......	43 000
1892	1.49	238 000	" "	......	8 000	......
1893	1.59	284 000	" "	10 000	44 000	......
1894	2.11	241 000	" "	10 000	1 000	......
1895	2.12	150 000	" "	......	......	80 000
Means of 5 years.......		220 000	230 000	4 000	10 600	24 600
1896	1.17	234 000	" "	......	4 000	......
1897	1.22	230 000	" "	......	0 000	0 000
1898	1.22	233 000	" "	......	3 000	......
1899	1.26	247 000	" "	......	17 000	......
1900	1.46	255 000	" "	......	25 000	......
Means of 5 years.......		239 800	230 000	0 000	9 800	0 000
1901	1.76	202 000	" "	......	......	28 000
1902	1.43	236 000	" "	......	6 000	......
1903	1.50	247 000	" "	......	17 000	......
1904	1.70	280 000	" "	20 000	30 000	......
1905	2.06	257 000	" "	20 000	7 000	......
Means of 5 years.......		244 400	230 000	8 000	12 000	5 600
1906	2.14	235 000	" "	10 000	......	5 000
1907	2.07	251 000	" "	10 000	11 000	......
1908	2.20	212 000	" "	......	......	18 000
1909	1.99	252 000	" "	......	22 000	......
1910	2.25	177 000	" "	......	......	53 000
Means of 5 years.......		225 400	230 000	4 000	6 600	15 200
1911	1.62	238 000	" "	......	8 000	......
1912	1.72	292 000	" "	30 000	32 000	......
1913	2.10	275 000	" "	30 000	15 000	......
1914	2.28	184 000	" "	......	......	46 000
1915	1.73	235 000	" "	......	5 000	......
Means of 5 years.......		244 800	230 000	12 000	12 000	9 200
1916	1.79	310 000	" "	40 000	40 000	......
1917	2.27	249 000	" "	30 000	......	11 000
1918	2.15	268 000	" "	30 000	8 000	......
1919	2.24	201 000	" "	......	......	29 000
1920	1.89	228 000	" "	......	......	2 000
Means of 5 years.......		251 200	230 000	20 000	9 600	8 400
1921	1.86	198 000	" "	......	......	32 000
1922	1.48	215 000	" "	......	......	15 000
1923	1.30	204 000	" "	......	......	26 000
1924	1.00	197 000	" "	......	......	33 000
1925	0.61	....	....	......	......	......
Means of 4 years.......		203 500	230 000	0 000	0 000	26 500
Mean of 34 years.......		233 700	230 000	7 100	8 900	12 300

From the volume of prime flow should be deducted 10 000 cu. ft. for Chicago. This leaves 220 000 sec-ft. for continuous water-power uses in the St. Lawrence River. This volume will be available every day of every year and in every desirable hour of every day. The Reservoir System of the Great

Lakes will guarantee the mass volume, and the forebay of Lake Ontario will absorb the minor irregularities of this water. The regulation of all the Great Lakes contributes 20 000 cu. ft. per sec. more of prime water than was determined by the Government engineers from the regulation of Lake Ontario alone.

The mean volume of water supply is precise to 2 per cent. It is an even chance whether it is 2% larger or 2% smaller. The mean value of the water supply for each 5-year period has a precision a little less than the mean flow of 34 years. For a single year, the possible error may be as much as 5%, sometimes too large, sometimes too small. The effect of these possible divergences is not cumulative.

The computations of level changes in the various reservoirs and forebays, as shown in Column (2), Table 2, have assumed in all the same depth of storage saving or loss. In practice, the storage in Lakes Erie and Ontario will be used only in low-supply years such as 1891, 1895, 1901, 1910, 1914, 1919, and 1924. In four years out of five, Lakes Erie and Ontario will be maintained at a high level with little surface variation. This will mean a gain of perhaps a foot in head for water-power uses and increased depths in the upper river channels, with beneficial effect on velocities and ice formation.

TABLE 3.—ST. LAWRENCE WATER SUPPLY AND YIELD OF GREAT LAKES.

Periods.	Total supply, in second-feet.	Yield per square mile of drainage basin, in second-feet.
1860-1864	241 000	0.838
1865-1869	252 000	0.876
1870-1874	232 000	0.807
1875-1879	249 000	0.866
1880-1884	260 000	0.904
1885-1889	235 000	0.817
Mean of 30 years	245 000	0.852
1891-1895	220 000	0.765
1896-1900	239 800	0.833
1901-1905	244 400	0.850
1906-1910	225 400	0.784
1911-1915	244 800	0.851
1916-1920	251 200	0.871
1921-1924	203 500	0.708
Mean of 34 years	233 700	0.813
Mean of 64 years	239 000	0.831

The mean regulated outflow values of the water supply of the Great Lakes and the yield in cubic feet per second for each square mile of tributary drainage basin are given in Table 3.

The supply diverted at Chicago after 1900 is counted as present in Table 3.

Gain by Regulation

It should be accentuated that the gain in electric energy from the increased outflow of 20 000 cu. ft. per sec. is prime power. The installation already estimated by the Government engineers is sufficient to use all the prime water all the time, with some margin for excess water and peak-load use. The increments of power from increased flow, using the same heads as the Government engineers, but using the coefficient 84% for over-all efficiency, are as follows:

Canadian:	Lachine Rapids........................	78 000 e. h. p.
	Coteau, Cedars, Split-Rock, Cascade Rapids	141 000 " " "
	Total, Canadian.....................	219 000 e. h. p.
International Section:	Long Sault Rapids.........	141 000 " " "
	Total, St. Lawrence River............	360 000 e. h. p.

The installation at the Long Sault contemplates 52 units, each rated at 32 250 e. h. p. on the station busses. At full gate, the turbines are estimated to use 4 580 sec-ft. The total output may be 1 680 000 e. h. p., and the maximum water use is 233 000 sec-ft.

It will be observed that the installation contemplated is sufficient for the continuous flow of 220 000 cu. ft. per sec., all used in the main power house at the Long Sault Rapids, without any diversion going to Massena. Under this assumption, the prime power to be developed in the Long Sault Rapids under a head of 74 ft. is 1 550 000 e. h. p.

The ultimate capitalized value of the additional 360 000 e. h. p., without additional installation, will not be less than $50 000 000.

Cost of Energy

The estimated cost of the initial power development, including all structures, canals, ship locks, regulating works at Rapide Plat, and channel dredging, is $159 197 200. The estimated operation and maintenance cost, including depreciation, is $1 782 000 per year. The unit cost is $103 per e. h. p., based on the prime volume of river flow. Let it be assumed that the cost may be $17 more per electric horse-power than estimated. This will care for some over-run, adjust matters with Massena, and take care of additional overheads.

With money at 5% and all other annual expenditures 3%, the possible rental price for the sale of electric power available every year for 365 days and every hour of every day is $9.60 per e. h. p.-year. In case the costs should prove to be $150 per prime e. h. p., the rental price would be $12 per e. h. p.-year. For the lesser cost, the sale price per kilowatt-hour, with 90% utilization, will be 1.63 mills and with 50% utilization, 2.94 mills. For the larger cost, the price per kilowatt-hour, with 90% utilization, is 2.04 mills; and with 50% utilization, 3.67 mills. These rentals will be competitive with those for Niagara Falls power, which the writer is informed, average about $15 per

e. h. p.-year, and for new contracts about $20 per e. h. p.-year, on the United States side.

With energy per electric horse power-year selling for $12, it is well to note the value of 1 ft. of additional head. It means 21 000 e. h. p. greater output for installations already paid for. This means not less than $200 000 per year added income, which, capitalized at 5%, shows a value of $4 000 000 per ft. of added head. Where additional head may be had for the expenditure of a less sum than this, it should be secured.

St. Lawrence Ice

The St. Lawrence has a bad name as a stream with ice complications. Frazil accumulates and drowns out lesser and sometimes major rapids. The back-water rise at the foot of the Long Sault Rapids is at times 15 ft. or more. It is believed this frazil formation is due to exposure in the rapids and in the shallow swift river and will terminate with the conversion of the river into a series of deep, still, slack-water pools. The writer has observed the vanishing of the ice in the deep, still St. Lawrence River above Ogdensburg, and in the deep, still St. Marys River above Sault Ste. Marie, Mich. The brilliant sunshine characteristic of late March and early April in these regions, and the temperature of the water mass itself, tend to honeycomb the ice. It breaks into needles and melts. It is spoken of colloquially as "rotting." The unreformed Niagara River is a more serious ice stream than the reformed St. Lawrence. In the Niagara many millions of dollars have been spent in hydro-electric construction, but up to the present time nothing has been done toward fundamental ice protection at Buffalo. Ice engineering is a phase of practice which will have to be given more attention in the future than in the past.

Market for Power

In order to visualize the meaning of the 1 550 000 e. h. p. to be initially developed in the International Section, it is well to compare other installations and other outputs. The allotment of each country will be about 880 000 e. h. p. It is stated* by F. A. Gaby, M. Am. Soc. C. E., Chief Engineer of the Ontario Hydro-Electric Power Commission, that, in 1924, the market for electric current in Ontario is 720 000 e. h. p.; that in 1926 there will be a market for 1 250 000 e. h. p., with a visible shortage of 150 000 e. h. p. in hydro-electric installation; and that the 1934 market will be about 2 650 000 e. h. p. This indicates a forecast of growth of 1 500 000 e. h. p. in 8 years, which is more than doubling the electric current consumed in this period. The market is tributary to both Niagara and St. Lawrence power. The 1924 Year Book of the Commonwealth Edison Company serving the Greater Chicago District shows 1 224 000 e. h. p. operating under a load factor of about 50% to be installed by 1925. The Year Book also shows that the Greater New York District had a maximum load in 1922 of about 1 418 000 e. h. p. The Niagara Falls Power Company of New York has a total installed capacity of 698 400 e. h. p., which is used mostly for electro-chemical and electro-metallurgical industries.

* *Journal*, Eng. Inst. of Canada, March, 1924, p. 14.

The growth in the absorbing capacity of various communities is very clearly indicated by the citations from Mr. Gaby. In the Greater Chicago District, the increase in consumption in 1923 over 1922 was over 15 per cent. The State of Illinois has an absorptive capacity of 1 e. h. p. per year for each 4 people of its population, Chicago and the surrounding territory within a radius of 150 miles for each 4.43 of population, and New York within a similar radius, for each 4.5 of population.

It should be understood that the region of Canada within 150 miles of the Long Sault Rapids is rich in water power and that comparatively few of the available sources have been developed. The Ottawa River entering the St. Lawrence River at Montreal has many available water-power sites; the St. Maurice entering the St. Lawrence at Three Rivers, 95 miles below Montreal, has much unused power and developments at Grand Mere, Shawinigan Falls, and La Gabelle. The extensive developments at Shawinigan Falls generate and transmit current for aluminum and public utilities, and grind wood pulp, with a down-hill rail haul of 21 miles to reach water navigation in the St. Lawrence. La Loutre Dam, recently constructed, creates a reservoir with a surface area of 300 sq. miles near the headwaters of the St. Maurice River and to some extent increases the low-water flow. It must be understood that the developments mentioned have the disadvantages of flashy streams, instead of the eternally even-running flow of the St. Lawrence River. The development of St. Lawrence power will flood the market and possibly degrade the values of many of these undeveloped sites.

On the United States side, it has already been demonstrated that a considerable block of power may be advantageously transmitted to New York State and to Southwestern New England. New York City is about equidistant from the Long Sault and from Niagara Falls—300 miles away. Schenectady is 160 miles from the Long Sault and 260 miles from Niagara Falls. Albany is 175 miles from the Long Sault, and Boston is 270 miles.

Prime Current

The very nature of the continuous even flow of the St. Lawrence River indicates the use of prime power for 24-hour industries operating 365 days a year and requiring electric current at a cost not exceeding 3 mills per kw-hr. The electric furnace is the dominant consumer. Perhaps the growth in the use of electric-furnace current has not been as rapid as that of public utilities in the past; but for the future it will serve an infinite variety of purposes.

Prime current permitting 100% utilization of great manufacturing plants is worth considerably more than intermittent current. In public utilities, prime current is worth about three times as much as excess current. For public utility uses, the cost of long-distance transmission lines indicates the wholesaling of base load current to various corporations operating in the region tributary to the St. Lawrence, while peak loads of short duration may be carried by local inexpensive steam installations. In the Greater Chicago District the electric current generated by steam averages 2.23 lb. of coal per kw-hr. of output. This means a fuel cost, at $5 per ton of coal, of 11.4 mills per kw-hr. This is a public utility, with a load factor of about 50%,

retailing about 60% of the energy for residential and commercial lighting and to small power users.

Canadian Policy

The energy to be generated in the Long Sault Rapids plant will be absorbed shortly after the development is completed. The remaining power of the St. Lawrence is Canadian. It will be an economic question for Canada to decide whether she will patiently await the inevitable coming of industries to her own territory with the inevitable building up of her commercial importance, or will transmit this power, with losses, to available markets largely in the United States. Questions of customs duties will enter, but the chances are that, in the end, the Mohamet of industry will be compelled to go to the Canadian mountain of power.

Niagara Water Power

The real immediate competitor of the St. Lawrence River power is Niagara River power, with a probable cost of production of less than 3 mills per kw-hr. The question is whether the great electro-chemical and electro-metallurgical industries will prefer first Niagara Falls or the vicinity of the Long Sault Rapids. Ultimately, no choice will exist; it must be Montreal.

Electric current from the Long Sault Rapids and from Niagara Falls will meet at Oswego, practically 135 miles from each development. The probable place of the interconnection of the two great hydro-electric systems will be at Syracuse, already reached by high-tension transmission lines from Niagara Falls, and substantially 150 miles distant from each of the developments.

Present Developments

The water power installations on the Niagara River aggregate about 570 000 e. h. p. for the American side and 700 000 e. h. p. for the Canadian side of the river. Only one of the present installations utilizes the water of the Niagara River under a head of over 300 ft. The total fall between Lake Erie and Lake Ontario is 326 ft. Between Lake Erie elevation at the Buffalo Lighthouse and the head of the Rapids approaching the cataracts at Niagara Falls, the difference in elevation is about 10 ft. This leaves a gross head of 316 ft., utilizable, with some slope losses, for hydro-electric development. (See Plate V.) The new Chippewa-Queenstown plant of the Hydro-Electric Commission of Ontario operates under a head of 304 ft. The head utilized by the Niagara Falls Power Company in its major installations on the New York side of the river is 212 ft. The effective head utilized by the Ontario plant of the Hydro-Electric Commission, taking water from near the head of the Rapids and discharging, with penstock losses, into the Maid-of-Mist Pool in the Gorge, is about 170 ft. The three other installations, one on the New York side and two on the Canadian side of the river, have effective heads of from 130 to 150 ft. These three plants are doubtless marked for relegation to peak-load and standby purposes, when new plants are built, to use efficiently the limited volume of flow.

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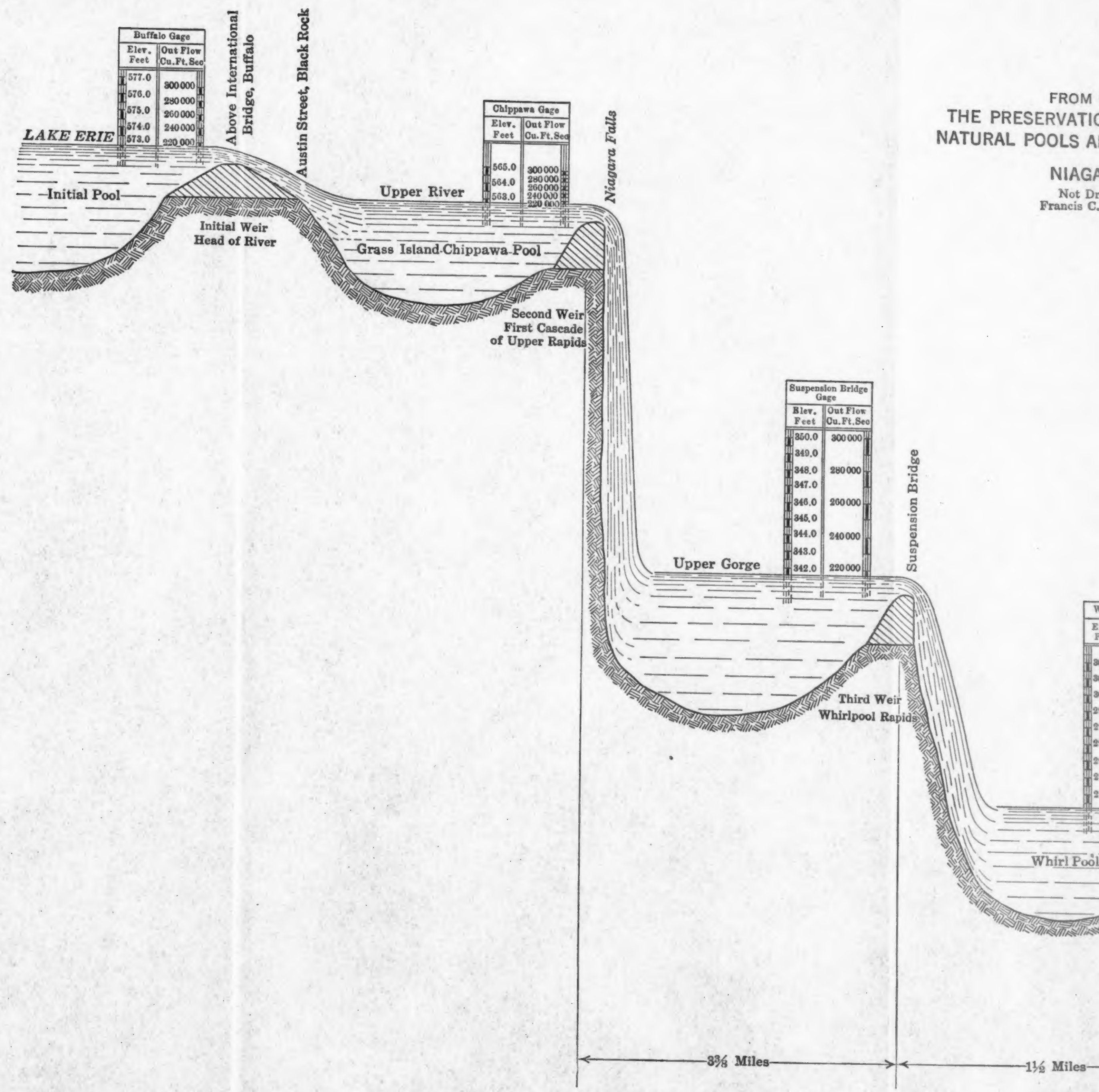
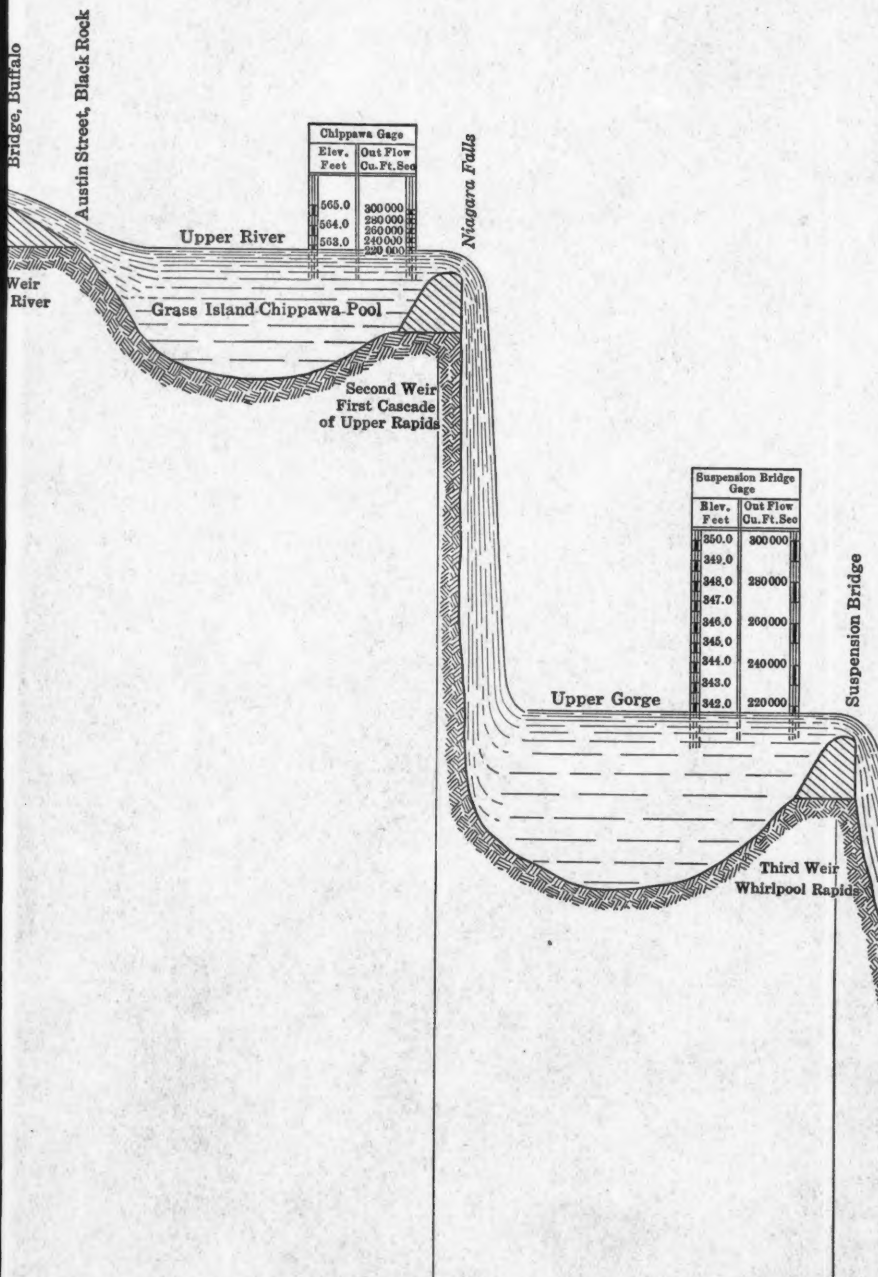




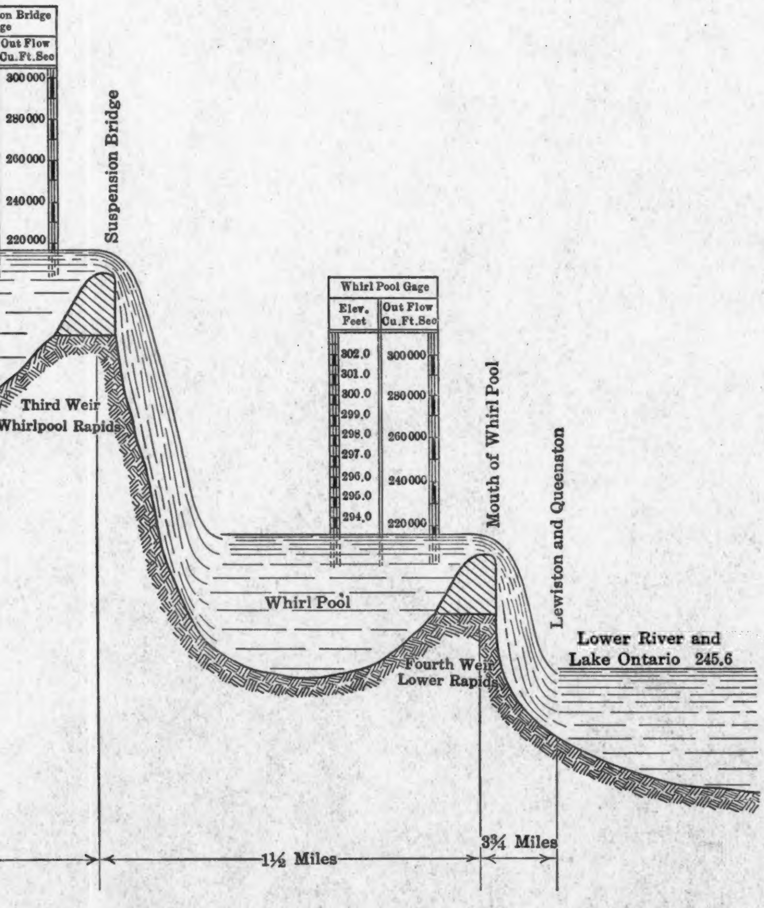
Bridge, Buffalo
Austin Street, Black Rock
Weir
River
Chippawa Gage
Elev. Feet
Out Flow Cu. Ft. Sec
565.0
564.0
563.0
300 000
280 000
260 000
240 000
220 000
Upper River
Niagara Falls
Grass Island-Chippawa Pool
Second Weir
First Cascade
of Upper Rapids
Suspension Bridge Gage
Elev. Feet
Out Flow Cu. Ft. Sec
350.0
349.0
348.0
347.0
346.0
345.0
344.0
343.0
342.0
300000
280000
260000
240000
220000
Upper Gorge
Suspension Bridge
Third Weir
Whirlpool Rapids
3⅜ Miles

PLATE V.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1925
SHENEHON ON
THE ST. LAWRENCE WATERWAY
TO THE SEA

FROM REPORT ON

THE PRESERVATION OF NIAGARA FALLS
NATURAL POOLS AND MEASURING WEIRS
OF
NIAGARA RIVER

Not Drawn to Scale
Francis C. Shenehon — 1908



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Treaty

The volume of flow for hydro-electric uses at Niagara Falls is limited, by the Boundary Waters Treaty of 1910, to 20 000 sec-ft. for the United States and 36 000 sec-ft. for Canada. The limitations imposed contemplate the preservation of the scenic grandeur of Niagara Falls. The combined water use of all present installations would exceed the limitations imposed by this Treaty. The use of water, therefore, in the more efficient plants must compel the stopping of the wheels in the less efficient plants.

Assuming for all developments at Niagara Falls an output of 29 e. h. p. for each cubic foot of water permissible under the Treaty of 1910 (56 000 sec-ft.), the total power output would be over 1 600 000 e. h. p. The actual output from the use of this volume of water is about 1 000 000 e. h. p., indicating that without reforming the treaty in quantity of water, but extending it to include diversions from the Maid-of-Mist Pool, an additional 600 000 e. h. p. will become available. This is 75% of the allotment of the United States at the Long Sault Rapids.

Future Power

It will be practicable to utilize, at Niagara Falls and in the Niagara Peninsula, after regulating works have been constructed, about 40 000 cu. ft. per sec. more water than was used in 1924 for all purposes of water power and navigation. Should this additional diversion become permissible under the terms of a new treaty with Great Britain, 1 160 000 e. h. p. more will become available. The efficient use of the 56 000 sec-ft. permissible, added to the proposed diversion, will make the ultimate volume about 2 800 000 e. h. p., which is 1 800 000 more than is now (1924) in use. As before stated, Niagara power, under a reformed treaty, is the greatest competitor of St. Lawrence power. The writer in a report on the preservation of Niagara Falls in 1908,* stated, "* * * the possibilities of continued and extended use of power at the Falls are conditioned upon the construction of regulating works in the Niagara River to avoid the wasteful outflow of the water of Lake Erie."

Power Uses

Before passing to the matter of the conservation and control of the waters of the Great Lakes for water-power betterment on the Niagara and St. Lawrence Rivers, it will be illuminating to note some uses of power at Niagara Falls: Abrasives, including carborundum; refractories, including fused alumina, high temperature electric furnaces, crucibles and pyrometer tubes; silicon carbide; firesand, electrically sintered magnesia; aluminum and its alloys in an infinite variety of forms; ferro-alloys, including ferro-silicon, ferro-chromium, ferro-tungsten, ferro-molybdenum and ferro-titanium; sodium bichromate and potassium bichromate; sodium; chrome alum, chromium sulphate, chromic acid, all three used in tanning leather; hydrochloric acid; carbon tetrachloride, sulphur chloride, caustic potash, caustic soda, liquid chlorine, bleaching powder, graphite, carbon electrodes, sodium chlorate, potas-

* Senate Doc. No. 105, 62d Cong., 1st Sess., p. 75.

sium perchlorate, barium chlorate, hydrogenated oils, titanium aluminum bronze, copper calcium alloy, potassium chlorate, formaldehyde; cyanamid, used as a fertilizer or for ammonia and nitric acid. It will be observed that these are all electric-furnace products. In addition, are the grinding of pulp wood and the making of paper with boilers electrically steamed for heating purposes, the grinding of wheat and grain, and an infinite number of small power uses.

Niagara Water Supply

The Niagara River, like the St. Lawrence River, has suffered seasons and years of low flow, because Nature appears to have little interest in the maintenance of the lakes for purposes of navigation, or the maintenance of an even flow in its rivers for water-power uses. The natural unrestrained outflow of the Great Lakes has served the elemental purpose of transferring from the basin of the Great Lakes to the ocean the eternally recurring rainfall surplus. For navigation and water-power uses, guidance by engineers is essential.

The water supply outflowing in a state of Nature through the Niagara River is shown in Table 4.

TABLE 4.—Niagara Water Supply and Yield of Great Lakes, Excepting Ontario.

Periods.	Total supply, in second-feet.	Yield per square mile of drainage basin, in second-feet.
1860-1864	202 000	0.793
1865-1869	213 000	0.836
1870-1874	202 000	0.793
1875-1879	206 000	0.809
1880-1884	234 000	0.919
1885-1889	210 000	0.786
Mean of 30 years	211 200	0.830
1891-1895	186 000	0.730
1896-1900	211 000	0.828
1901-1905	210 000	0.824
1906-1910	198 000	0.777
1911-1915	218 000	0.856
1916-1920	215 000	0.844
1921-1924	175 000	0.687
Mean of 34 years	202 600	0.796
Mean of 64 years	206 700	0.811

In Table 4, the outflow since 1900 in the Drainage Canal at Chicago is counted as outflowing in the Niagara River. The volumes for navigation passing through the Welland and Erie Canals, and the volume passing through the Welland Canal for water-power uses, are likewise counted as part of the water supply shown.

Comparing Table 4 with Table 3, showing the water supply of the St. Lawrence River, it will be observed that the earlier 30 years and the later 34 years, show very little change in supply, demonstrating that the water supply of the Great Lakes is not failing. The indication is that changes at the Galops Rapids have modified the St. Lawrence outflow. Based on this comparison of the supply of the two streams, the water supply for the St. Lawrence River has been adjusted by subtracting an obviously excessive quantity. It will be observed that the local supply of Lake Ontario for the mean of 64 years, amounts to about 32 000 cu. ft. per sec. more than the Niagara supply. The yield for Lake Ontario is somewhat in excess of its percentage drainage basin area.

Gain by Regulation

The result of utilizing all the Great Lakes as reservoirs shows a gain of 20 000 sec-ft. of dependable flow for the Niagara River. After deducting 10 000 cu. ft. per sec. for the Chicago diversion, the mean supply for a period of 64 years is 196 700 cu. ft. per sec. With the proper limitations in the pull-down of Lakes Superior, Michigan-Huron, and Erie, the volume of prime outflow may be taken as 190 000 cu. ft. per sec. If 100 000 cu. ft. per sec. be assigned to water power and navigation uses in the Niagara River and in the Niagara Peninsula, 90 000 cu. ft. per sec. remains available to perpetuate the scenic grandeur of Niagara Falls.

Preservation of Niagara Falls

Hydro-electric engineers do not always accept graciously the thought that a great volume of water which might serve industry, be devoted to the maintenance of a great spectacle. Perhaps the fate of the cataracts may be forecast by saying that when the poverty of the nation compels the conversion of the National Parks, including vast timbered tracts, to commercial uses, and when money is no longer spent for city parks and playgrounds, and architectural refinements in buildings are omitted, and the art institutes are closed—then all the water of the Niagara, as far as practicable, may be devoted to hydro-electric uses. For the present, not less than 90 000 cu. ft. per sec. of water must be reserved for the preservation of the scenic grandeur of Niagara Falls.

Value of Added Water

The value of 20 000 sec-ft. of dependable water added to the Niagara River as a result of Great Lakes regulation may be arbitrarily assigned at $58 000 000. It should be stated here that the value of water power at Niagara Falls or on the St. Lawrence is not the difference in cost between electric energy generated by steam or water. The reason is that industries requiring cheap power would seek other water powers or use steam installations near the pit mouth. The market for the Niagara and St. Lawrence is created by the cheapness of the power. It should be observed also that there is no vital concern in regard to the saving of bituminous coal, because at the present rate of consumption, the visible supply in the United States will last for over 5 000 years.

Great Lakes Regulation

The writer has repeatedly referred to Great Lakes regulation. It is desirable to visualize clearly the Great Lakes System. It is made up of two immense reservoirs, Superior and Michigan-Huron, with little or no water-power development in the outflowing rivers, St. Marys and St. Clair. The Niagara River, with its great descent of 326 ft., has immediately above it the lesser reservoir of Erie as a forebay; and likewise the St. Lawrence River with its power drop of 200 ft. has above it the lesser reservoir of Ontario as a forebay. (See Fig. 7.)

The writer in 1923 had a part in the development of hydro-electric power on the western slope of the Sierras in the Eldorado Region of California. Terms used there are applied to the Great Lakes for clearer visualization. Near the crest of the Sierras, storage reservoirs conserve the rainfall and the melted snow of the wet season for use during the dry season. Reservoir water passes through ditches, canals, and flumes, to a smaller reservoir, called a forebay, having a supply good for 12 to 120 hours, immediately above the 1 800-ft. power drop. The forebay is used for peak-load operation and for emergency supply in case of interruption in ditch service. The water of the reservoirs and of the forebay is expended on a well planned budget system. On the Great Lakes, Superior and Michigan-Huron are the reservoirs in the High Sierras and Erie and Ontario are the forebays just above the power drops. Erie will serve magnificently as a forebay for Niagara power and will permit variations of flow in the Niagara River during the different hours of the day. Any interference with navigation in the Niagara River will be trivial compared to the vast economic gain from intermittent river flow. In the same manner, Ontario will serve magnificently as a forebay for St. Lawrence power, provided the developments in the St. Lawrence are made so as to constitute an uninterrupted series of slack-water pools.

Ontario Regulation

Beginning with Lake Ontario, it has already been stated that regulating works are a part of the project recommended by the International Joint Commission. The detail of these works is immaterial in the present discussion. The principles of budget outflow alone need consideration.

The writer suggests that Lake Ontario be regulated with a pull-down not much over 1 ft., aiming at Elevation 247.0 at the beginning of each year. This will be good for navigation. No danger of high water will exist in Lake Ontario, because any large local supply such as occurred in the spring months of 1912 and 1913, will be offset by checked flow in the Niagara and St. Clair Rivers, or by the discharge of an excess volume to the sea through the St. Lawrence River.

It is desirable for purposes of control to increase the channel capacity of the St. Lawrence River to the extent of 50 000 cu. ft. per sec. This will mean no more dredging than will be required for the utilization of the highest head without too great slope losses, and for the efficient solution of the frazil ice

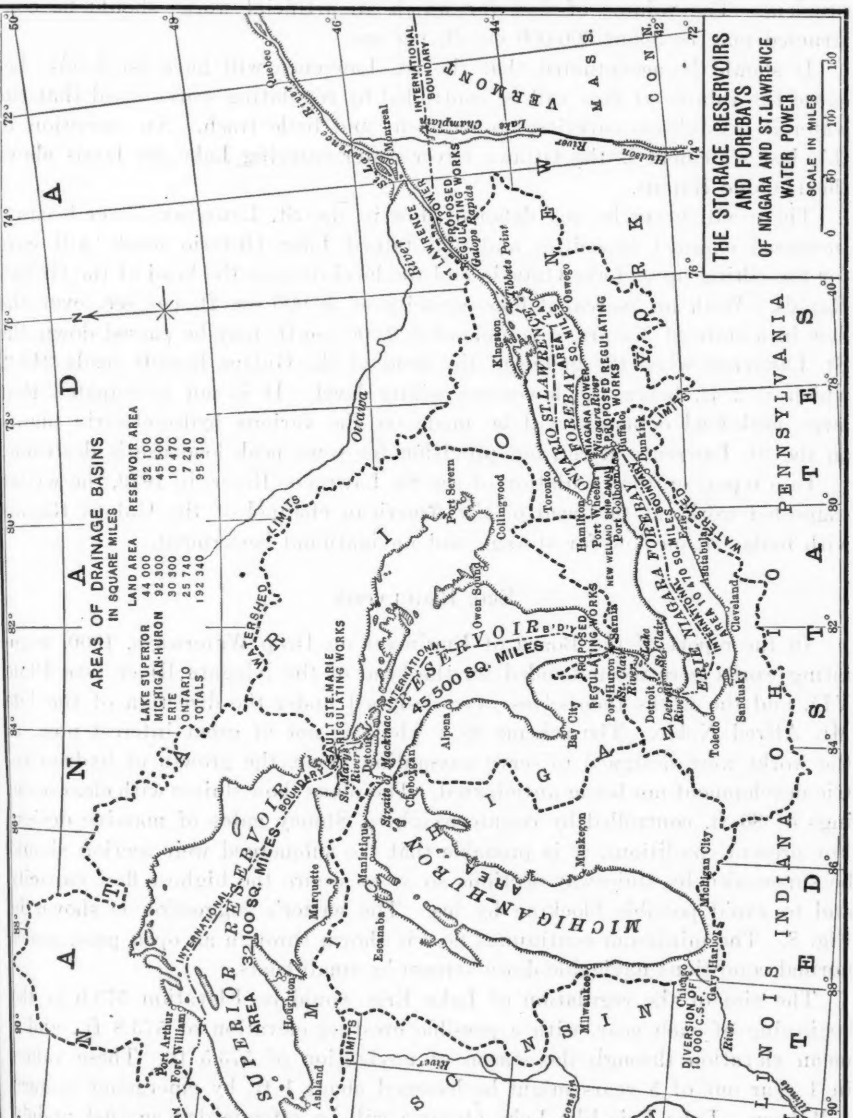
THE STORAGE RESERVOIRS AND FOREBAYS OF NIAGARA AND ST. LAWRENCE WATER POWER
SCALE IN MILES
0 50 100 150
AREAS OF DRAINAGE BASINS IN SQUARE MILES
LAND AREA RESERVOIR AREA
LAKE SUPERIOR 44 000 32 100
MICHIGAN-HURON 92 300 45 500
ERIE 30 300 10 470
ONTARIO 25 740 7 240
TOTALS 192 340 95 310
SUPERIOR RESERVOIR AREA 32 100 SQ. MILES
MICHIGAN-HURON RESERVOIR 45 500 SQ. MILES
ONTARIO ST. LAWRENCE FOREBAY AREA 7 240 SQ. MILES
ERIE NIAGARA FOREBAY AREA 10 470 SQ. MILES
DIVERSION 10 000 C.S.

FIG. 7.

problem. The volume of flow for which all artificial works should be constructed may be about 300 000 cu. ft. per sec.

It should be accentuated that the St. Lawrence will have no floods—because the volume of flow will be controlled by regulating works—and that the water is crystalline, carrying no sediment and little trash. An exception to this last statement is the Ottawa River water entering Lake St. Louis above the Lachine Rapids.

There will never be any deficient flow in the St. Lawrence River because increased channel capacities and maintained Lake Ontario levels will care for the tilting up of Lake Ontario and the level drop at the head of the Galops Rapids. With an increased flow capacity of 50 000 cu. ft. per sec. over the flow in a state of Nature, a volume of 220 000 sec-ft. may be passed down the St. Lawrence when the gauge at the head of the Galops Rapids reads 242.2, which is 2 ft. below the low-water winter level. It is not anticipated that large peak-load demands will be made on the various hydro-electric plants on the St. Lawrence River, but provision for some peak load use is desirable.

In a report on the regulation of the St. Lawrence River, in 1919, the writer suggested works at the head of the American channel of the Galops Rapids with budgeted outflow for storage and navigational betterment.

Erie Regulation

In the report of the Board of Engineers on Deep Waterways, 1900, regulating works were recommended for the head of the Niagara River (see Plate VI), and the works themselves were designed under the direction of the late Mr. Alfred Noble. The scheme as a whole is not of great interest now, as the works were designed to serve navigation alone, the growth of hydro-electric development not being anticipated. The plans show sluices with clear openings of 80 ft. controlled by counterweighted Stoney gates of massive design. For present conditions, it is probable that the submerged weir section should be superseded by sluiceway sections so as to secure the highest flow capacity and to avoid possible blocking by ice. The writer's suggestion is shown in Fig. 8. The minimum continuous flow is shown through an open pass, under normal conditions navigable down stream by small boats.

The aim of the regulation of Lake Erie would be Elevation 573.0 at the beginning of each year, with a possible cresting elevation of 573.8 ft., and a mean elevation through the season of navigation of 573.5 ft. These values in 1 year out of 5 years might be lessened about 1 ft. by emergency storage pull-down. Lake Erie like Lake Ontario will be safeguarded against unduly high water by the checking of the St. Clair River flow, or by sending excess quantities to Lake Ontario and through the St. Lawrence to the sea.

Ice in the Niagara River

The ice problem at the head of the Niagara River and violent storm conditions, with waves battering against the regulating works, need careful consideration. The writer suggests that ice should preferably rot in Lake Erie, as it rots at the head of the St. Marys and in the St. Lawrence, with no heavy

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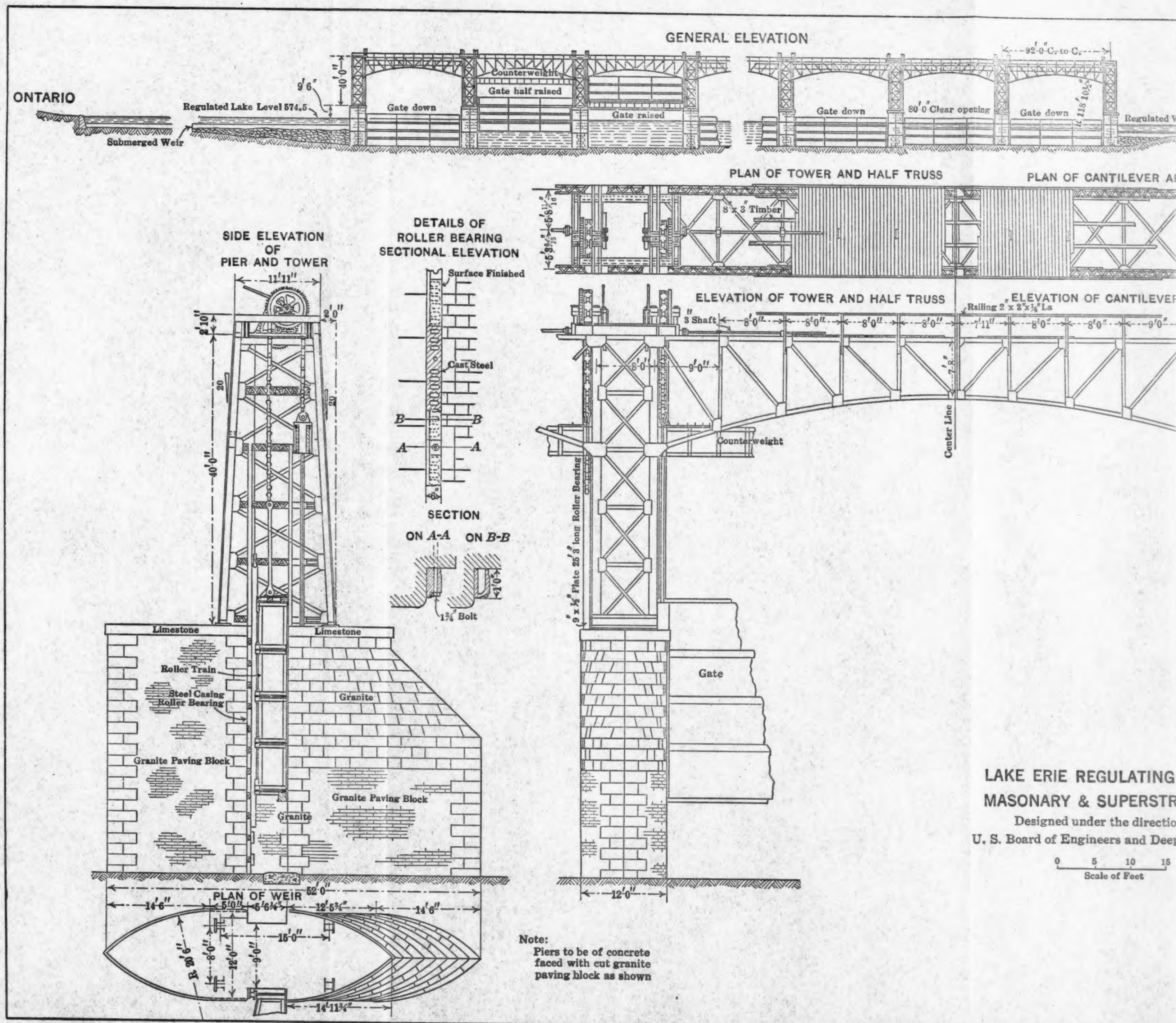


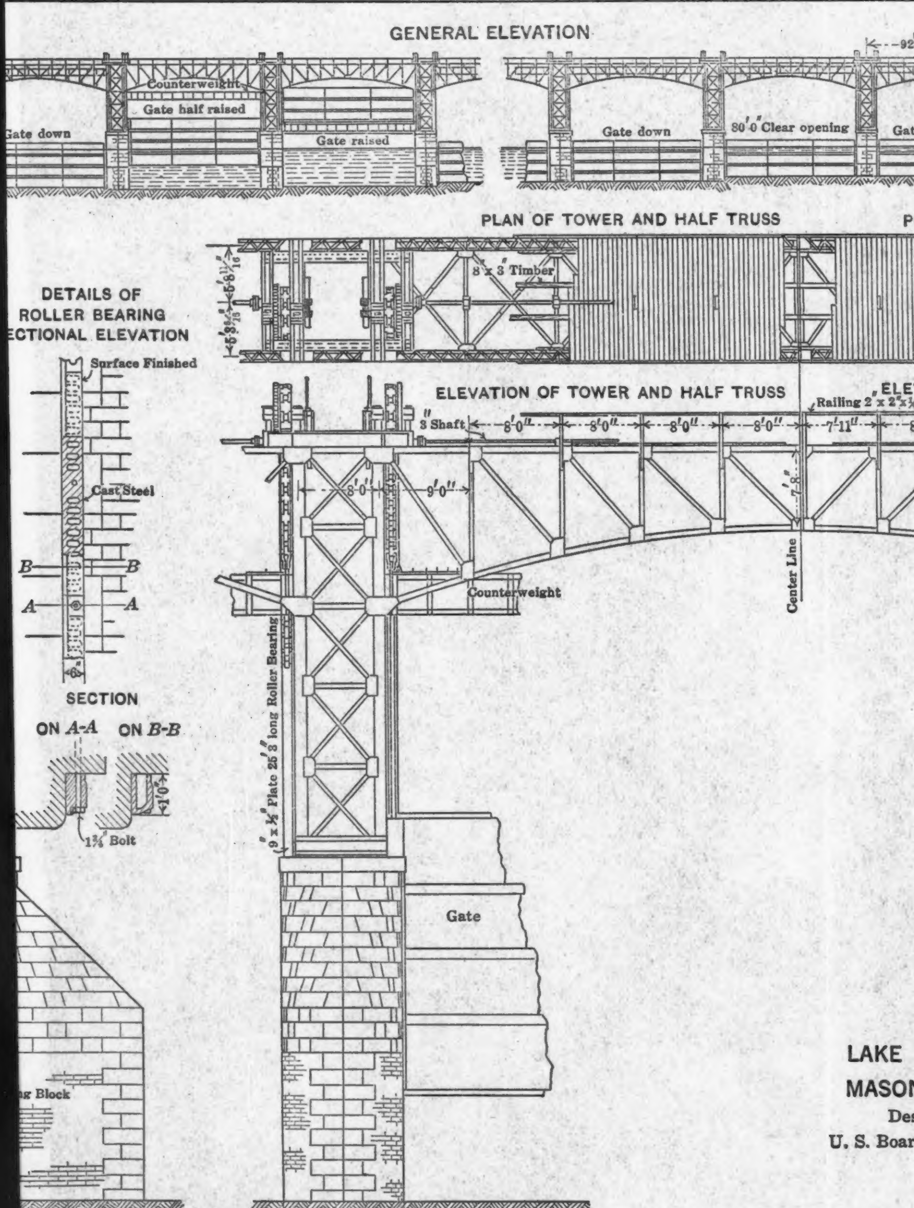
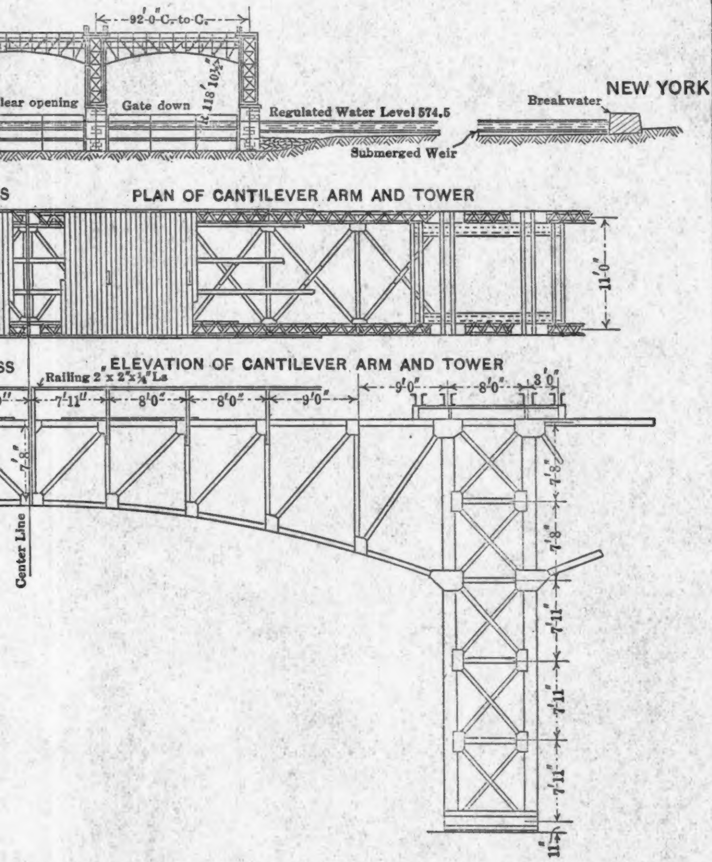

GENERAL ELEVATION
Counterweight
Gate half raised
Gate down
Gate raised
Gate down
80'0" Clear opening
PLAN OF TOWER AND HALF TRUSS
8" x 3" Timber
DETAILS OF
ROLLER BEARING
ECTIONAL ELEVATION
Surface Finished
Cast Steel
ELEVATION OF TOWER AND HALF TRUSS
Railing 2" x 2"
3" Shaft
8'0"
7'11"
9'0"
Center Line
Counterweight
9" x 1/2" Plate 25' 3" long Roller Bearing
SECTION
ON A-A
ON B-B
1 3/4" Bolt
1'0"
Gate
12'0"
14'6"
Note:
Piers to be of concrete
faced with cut granite
paving block as shown
LAKE
MASON
U. S. Boar

PLATE VI.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1925
SHENEHON ON
THE ST. LAWRENCE WATERWAY
TO THE SEA



LAKE ERIE REGULATING WORKS
MASONARY & SUPERSTRUCTURE
Designed under the direction of
U. S. Board of Engineers and Deep Waterways

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Scale of Feet

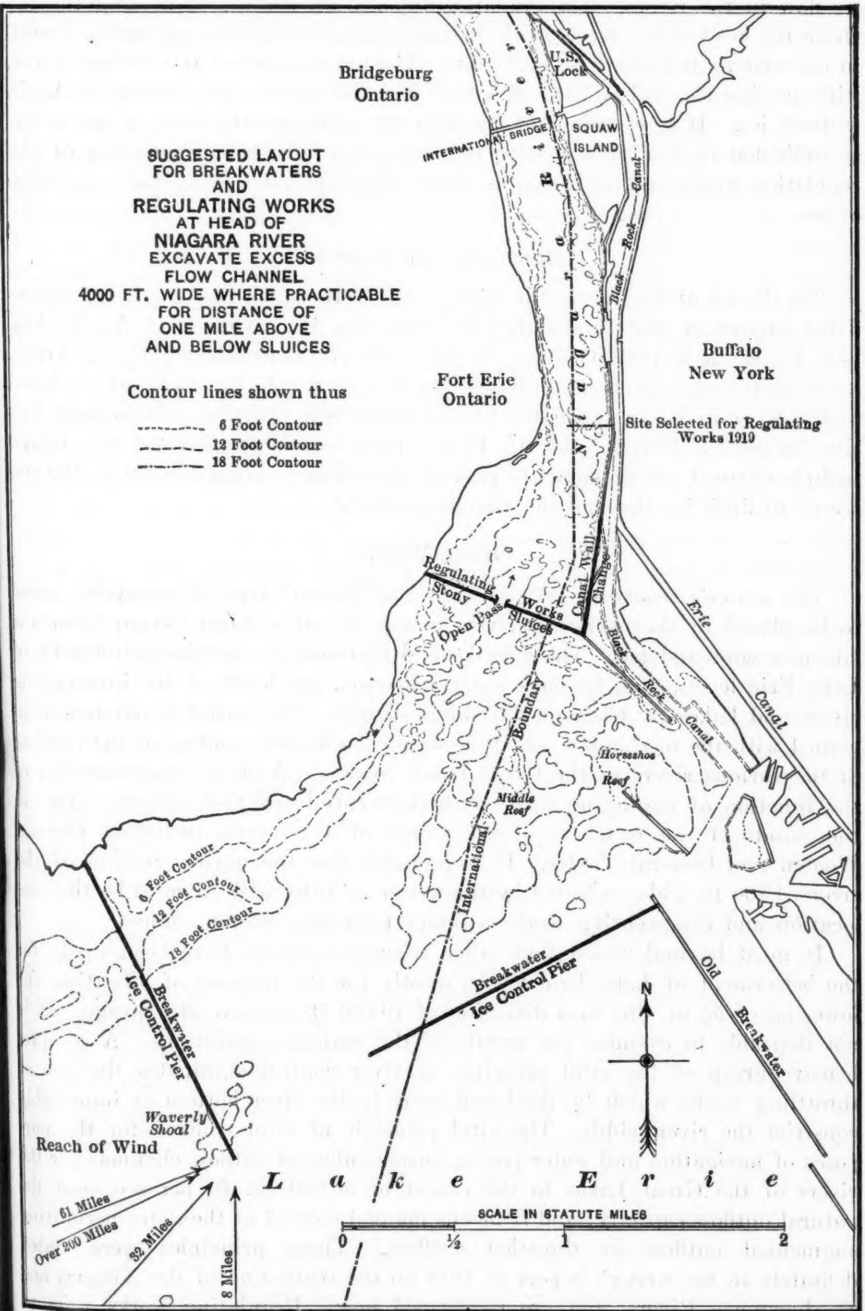
SUGGESTED LAYOUT
FOR BREAKWATERS
AND
REGULATING WORKS
AT HEAD OF
NIAGARA RIVER
EXCAVATE EXCESS
FLOW CHANNEL
4000 FT. WIDE WHERE PRACTICABLE
FOR DISTANCE OF
ONE MILE ABOVE
AND BELOW SLUICES
Contour lines shown thus
6 Foot Contour
12 Foot Contour
18 Foot Contour
Bridgeburg
Ontario
U.S. Lock
INTERNATIONAL BRIDGE
SQUAW ISLAND
Black Rock Canal
Niagara River
Buffalo
New York
Fort Erie
Ontario
Site Selected for Regulating Works 1919
Regulating Works
Stony
Open Pass
Sluices
Canal Wall
Change
Erie Canal
Black Rock Canal
Horseshoe Reef
Middle Reef
International Boundary
6 Foot Contour
12 Foot Contour
18 Foot Contour
Ice Control Pier Breakwater
Breakwater
Ice Control Pier
Old Breakwater
N
Waverly Shoal
Reach of Wind
61 Miles
Over 200 Miles
32 Miles
8 Miles
Lake Erie
SCALE IN STATUTE MILES
0 ½ 1 2

FIG. 8.

ice flow in the rivers. The climatic influence on Buffalo of holding the ice above the head of the river, while it honeycombs, breaks up, and melts, would, in the writer's judgment, be negligible. The temperature of the surface water, with melting ice, will be little different from the surface temperature in April, without ice. It is certain that breakwaters enclosing the head of the river, as indicated in Fig. 8, will solve the question of the storm battering of the regulation works and, at the same time, will eliminate largely the congestion of ice.

Recommended Works

The Board of Engineers for Rivers and Harbors, U. S. Army, in a report dated August 24, 1920, and signed by Brig.-Gen. H. Taylor, U. S. A., M. Am. Soc. C. E., now (1925) Major General, Chief of Engineers, U. S. Army, has recommended the immediate construction of regulating works at the head of the Niagara River, estimated to cost $8 000 000, utilizing substantially the site chosen by Alfred Noble in 1900. The recommendations of the Board include channel enlargements to provide for a flow of from 300 000 to 400 000 sec-ft. to flush ice through the Rapids Section.*

Other Works

The writer's report of 1919 suggested a "daring" type of removable gates to be placed in the narrowest river section, 1½ miles down stream from the site now contemplated. These works had in mind the definite betterment of Lake Erie levels, and, by back-water reflection, the levels of the intervening rivers and Lakes St. Clair and Michigan-Huron. The writer is not now concerned with the mechanism which accomplishes flexible control of the outflow of the various rivers in the Great Lakes System. A clear comprehension of the function of regulating works is first essential and that already exists in the minds of the members of the Corps of Engineers, including Colonel Wooten and General Taylor. It is probable that the narrow section of the river—1 800 ft. wide—selected by the writer in 1919, will prove to be the best location and disappearing works a better type than Stoney sluices.

It must be understood that other recommendations have been made for the betterment of Lake Erie levels, mostly for the purpose of offsetting the lowering of 5½ in. due to a diversion of 10 000 ft. per sec. at Chicago. It is not desirable to examine the merits of the various suggestions. A comprehensive grasp of the vital principle of river control eliminates the use of throttling works which lie dead and inert in the river bottom or immovably constrict the river width. The vital principle of river control, for the purposes of navigation and water power, means enlarged outflow channels for the rivers of the Great Lakes to the extent of 50 000 cu. ft. per sec. over the natural outflow capacity; and it means manual control of the gates permitting augmented outflow or throttled outflow. These principles were stated definitely in the writer's report of 1919 on the regulation of the Niagara and St. Lawrence Rivers, and are re-affirmed now. Regulating works refer to

* Report on "Diversion of Water from the Great Lakes and Niagara River," Washington, D. C., 1921.

works manually operated for elastic control; compensating works refer to inert submerged weirs or wing dams.

Intermittent Flow

In addition to the control of Niagara River flow for navigation and water power, is the problem of reasonable fullness in the cataracts and rapids during a portion at least of each day from May to October. The writer proposes for the cataracts a daylight volume of 150 000 sec-ft. during the months mentioned. While this large natural flow goes over the cataracts, water-power uses will be cared for by mill-pond water accumulated in the Erie forebay during 16 hours of diminished cataract and rapids flow. This may mean a river flow of 250 000 sec-ft. for 8 hours, and of 160 000 sec-ft. for 16 hours per day.

It is probable that it will be economical to build control gates at the head of the rapids above the cataracts to maintain a higher level in the Chippewa-Grass Island Pool. This will give greater depths in the water-power canals, with a gain in effective head, and also deeper navigable channels during the periods of lessened flow. These gates will serve also to re-distribute the flow over the crest of the Horseshoe Falls, lessening destructive action at the apex, and increasing depths at the extremities of the crest line. Re-distribution of the flow in the Horseshoe Falls was recommended in the writer's report of 1908, already referred to.

It should be observed lest it appear that this discussion is far afield from the St. Lawrence Waterway to the Sea that the regulating works recommended in 1900 were a part of all deep waterways to the sea, including that of the St. Lawrence to the foot of Lake St. Francis and thence through Lake Champlain to the Hudson River and New York. The maintenance of navigable depths in the lakes and rivers and the best utilization of the water supply of the Great Lakes are dominant elements in the St. Lawrence Waterway to the Sea.

Michigan-Huron Regulating Works

The general layout of regulating works at the head of the St. Clair River, to improve navigable depths in Lake Michigan-Huron and in St. Marys River to the foot of the rapids, including better depths over the lower breast walls of the ship locks at Sault Ste. Marie, is shown in Fig. 5; and these works, with the appurtenant navigable passes, have already been briefly described. The economic desirability and the physical practicability of the regulation of these immense lakes are certain. The same principles apply as in the case of the St. Lawrence and Niagara Rivers, that additional outflow capacity is desirable and that the regulating works shall give elastic manual control. (See pages 1260 and 1298.)

Over-Emphasizing Minor Losses

For the past 25 years, during the season of navigation, Lakes Michigan-Huron have been from 18 to 40 in. lower than desirable. Since the opening of the canal of the Sanitary District of Chicago in January, 1900, however,

more concern has been felt for the loss of level, increasing during the period from 2 in. to 5 in., than for the large degradation of levels coming from other causes, and engineers have given their attention to the reclamation of the 5 in. rather than to the larger needs. It is obvious that the lesser loss could be compensated by contraction works in the form of submerged weirs or wing dams. A Board of United States Engineers reported in 1914 on a series of submerged weirs in the St. Clair River to restore 6 in. The writer proposed in 1919 to raise Lakes Michigan-Huron, as did the Board of Engineers on Deep Waterways in 1900, by back-water coming from a super-elevation of Lake Erie. The International Waterways Commission, 1910, gave consideration to the regulation of Lakes Michigan-Huron, but reached a conclusion that the natural regulation of the lakes was so perfect that it could not be improved on by the works of man.*

Mr. Grunsky's Forecast

"The Low Stage of Lakes Huron and Michigan" has been discussed in a paper† by C. E. Grunsky, President, Am. Soc. C. E. Speaking of the increased elevation of Lakes Michigan-Huron through back-water from the super-elevation of Lake Erie, Mr. Grunsky states:‡

"It is possible, moreover, to go a step farther and to provide works for the throttling of the waterway, or for the complete control of flow, within fixed limits, of the St. Clair River. The stage of Lakes Huron and Michigan could then be brought under adequate control, and the lake stage could be maintained at all times as high as required by navigation interests.

"That regulating works will sooner or later be constructed, in the rivers draining the several lakes, of such a character that the elevation of the lakes under proper management cannot fall below predetermined minima, is reasonably certain. It follows, therefore, in view of the benefit that will be derived from the use of Lake Superior as a storage basin, and of the control of water surface that will result from works below Lake Huron, that navigation interests will be but temporarily, if at all, affected by a diversion of water at Chicago, or elsewhere within reasonable limits as to amount, and that any deleterious effect will not continue beyond the time when the water elevation of this lake and Lake Michigan will be controlled by works below Lake Huron.

"The stages of Lake Ontario and of the St. Lawrence River are questions apart from the one that has here been discussed. Enough has been said, however, to show that here, too, the less than normal stages of recent years are to be ascribed to climatic conditions, supplemented in a very slight degree only by the diversion at Chicago. The ultimate effect of the diversion of water from any of the lakes, offset by the equalizing effect of lake regulation, upon the stage of water in the St. Lawrence River is a study that will have to be made when the regulating works come under consideration."

Superior Regulation

The writer's suggestion now is to regard the regulation of the lakes, not as piecemeal and individual, but as a continuous, interrelated series in which each lake serves all the others. As Lakes Superior and Michigan-Huron are treated as a unit of storage, it is well to pass immediately to the existing

* Report of the International Waterways Comm. on the Regulation of Lake Erie, 1910.

† *Transactions*, Am. Soc. C. E., Vol. LXIII (June, 1909), p. 31.

‡ *Loc. cit.*, pp. 46–47.

regulating works at the head of St. Marys River, giving manual control of the outflow of Lake Superior. The original design of these controlling works was made under the direction of the late Mr. Alfred Noble, very shortly after the design of the works at the head of the Niagara River. Sections of the works were built to compensate for water-power diversions in lateral canals, and these sections existed for a number of years as dead, inert, unoperated, compensating works, permitting diversions without unduly disturbing the level of Lake Superior. Later, water-power development on both sides of the river became the adequate reason for completing the works and operating them manually. These works now consist of a battery of sixteen sluices, each with a clear opening of 52.2 ft., with massive steel Stoney gates. In addition, there are three 33-ft. sluices in the dam of the United States power plant. These works are shown in Figs. 4 and 10. The original construction is detailed in a paper by G. F. Stickney, M. Am. Soc. C. E., entitled "The Compensating Works of the Lake Superior Power Company."* The recent works and their operation have also been described† in much detail by Louis C. Sabin, M. Am. Soc. C. E.

It will be assumed that the regulating works, with the added flow in the water-power canals on each side of the river, have an excess outflow capacity of 50 000 sec-ft., or more, over that of the natural rapids. The volume of flow through the wheels, either running or blocked open, may be regarded as effective outflow capacity for the water-power plants on the St. Marys River. In considering the capacity of flood flow on flashy streams where abnormal conditions of ice or trash might prevent penstock flow, it is regarded as desirable to have other spillways of adequate capacity. For the streams of the Great Lakes, however, no comparable flood conditions exist. A brief shutdown of the power plants at Sault Ste. Marie at the time of maximum supply, will have a negligible effect on the level of Lake Superior. On the Niagara and St. Lawrence, however, the well-being of the tandem series of plants should involve adequate spillway capacity so that lower plants may not be shut down in case of blockade in an upper plant.

It is noteworthy that no scheme of regulation to better navigable conditions by the maintenance of higher surface levels has originated with the shipping interests of the Great Lakes. With the exception of the Gut Dam at the Galops Rapids of the St. Lawrence, the compensation has always been suggested by diversions for sanitary or water-power purposes, rather than for the maintenance of levels when the supply conditions of the Great Lakes needed the throttling of the outflow toward the ocean.

The water power at Sault Ste. Marie contemplates a prime diversion of 50 000 sec-ft., with the use of 10 000 sec-ft., or more, of excess water when Lake Superior supply warrants additional outflow. The use of this water for power purposes at Sault Ste. Marie, under a head of 20 ft., less slope losses in the canals, should amount to about 90 000 e. h. p. The actual development

* *Transactions*, Am. Soc. C. E., Vol. LIV (June, 1905), p. 346.

† In an article in the *Military Engineer*, reprinted in the *Canadian Engineer*, March 21, 1920.

is somewhat less than this. The water power here, with perhaps an effective head of 18 ft., is not comparable with the great powers at Niagara with a head of over 300 ft., or with the ultimate great powers on the St. Lawrence with heads aggregating 200 ft. The release of Lake Superior water should contemplate the well-being of navigation and the lower water powers, rather than the over-accentuation of the local water powers of St. Marys River. The water power at Sault Ste. Marie, is largely used for the making of carbide, ferro-alloys, electric smelting, the grinding of wood pulp, and the making of paper.

Assuming a cost of $750 per ft. for works installed at Sault Ste. Marie, under post-war conditions, the total cost for the main river section is about $750 000. The cost on the American side was borne initially by the water power companies, but the United States is assuming the cost by the remission of water power rentals. It should be understood that at Sault Ste. Marie, under the decision in the famous Chandler-Dunbar case, all submerged lands were taken over by the Federal Government for purposes of navigation. This made the United States a water power proprietor, receiving each year as rentals for the use of the water, $2 per sec-ft. for prime water and $1 for secondary water. Here, then, exists a little subsidization of navigation by water power.

The operation of the regulating works in St. Marys River is under an International Board, the program contemplating the maintenance of Lake Superior levels between the limits of 601.5 and 604.1 ft. The program is based on the needs of Superior alone without necessary benefit to Michigan-Huron.

The final closing of the gap completing the work and placing Superior under full manual control is so recent that the value of regulation is not yet fully demonstrated. In the paper already referred to, Mr. Sabin writes:

"When the works are entirely completed the flexibility of the system will be increased, since the entire flow will be under control; the degree of regulation will not be materially improved, however, but a greater use of water for power will be permissible. What will be accomplished, then, is the continuous use of the natural minimum flow, equal to about 75% of the mean discharge of the river, and a reduction of the fluctuation in monthly mean levels from a range of 3.8 ft. to a range of about 2.6 ft. The economic advantage of power development to the extent of some 80 000 h. p. is thus combined with improvements of navigation conditions."

Regulated Levels

Lake Superior has not been entirely immune from the low-water régime existing on the other lakes. The reason for this is the tremendous inertia of the lake, which will require a number of years of ample supply or restricted outflow to bring it up. Assuming that it is desirable to raise the surface level of Lake Superior 1 ft. over the present level, it will be necessary to throttle the outflow to the extent of over 28 000 sec-ft. for 1 year, or 14 000 sec-ft. for 2 years. At a time when the lower lakes need the water, it is not practicable to accomplish this restoration of Lake Superior level. This must await regu-

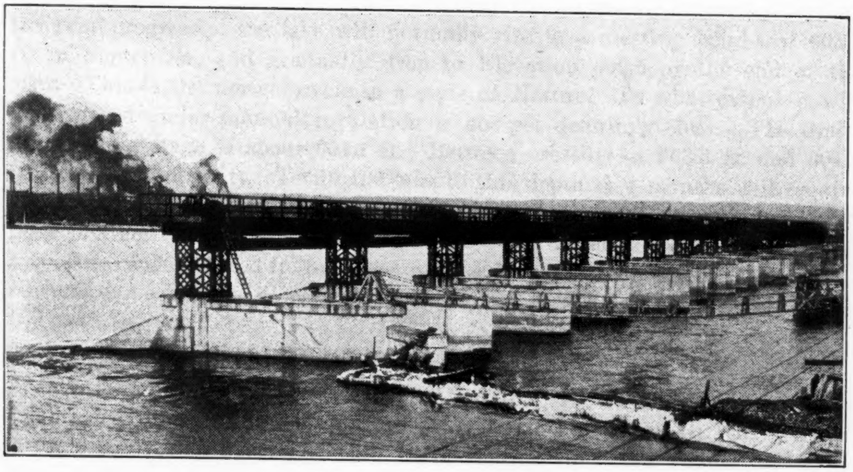

FIG. 9.—VIEW OF REGULATING WORKS IN ST. MARYS RIVER.

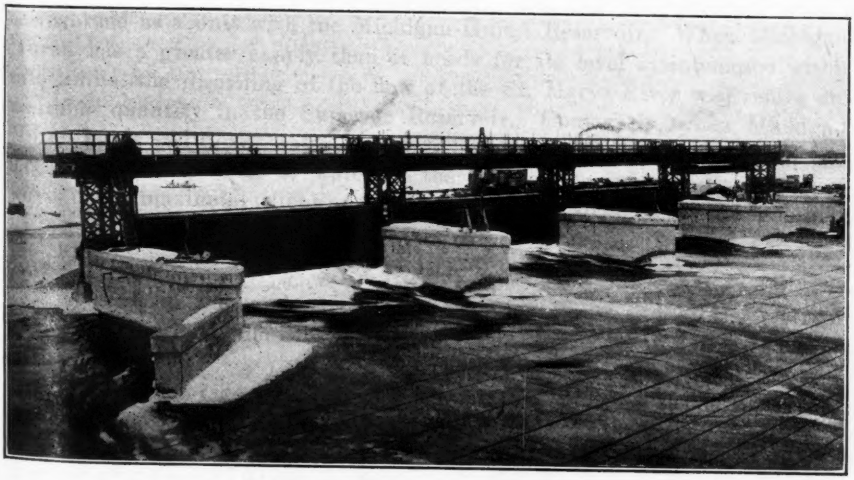

FIG. 10.—VIEW OF PART OF REGULATING WORKS AT SAULT STE. MARIE.

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lating works in all of the Great Lakes, so that the supply may be simultaneously throttled in all.

In the scheme of regulation suggested by the writer, Lake Superior will preferably stand at about Elevation 602.5 at the beginning of each year. As the year progresses, the lake will normally rise to a cresting height of 603.3 ft. in September, and gradually drop to Elevation 602.5 by the end of the year. This is the normal cycle in a state of Nature. To what extent it will be modified under manual regulation is not yet definitely clear. The limiting cresting stage is about 604.1 ft. Between cresting at 603.3 ft. and 604.1 ft. is a depth of 0.8 ft. To fill the lake to this depth in 4 months will require an excess of roundly 85 000 sec-ft. more than the normal generous supply of these productive months. As the excess capacity of the outflow works permits a release of 50 000 sec-ft. over the release in a state of Nature, the actual rise might be held to Elevation 603.6.

Effect on Michigan-Huron

The dumping of 50 000 sec-ft. for 4 months will raise the level of Lakes Michigan-Huron, with manual control at the head of the St. Clair River, about 0.4 ft. The hazard of high water in Superior, with regulating works in operation and with 50 000 sec-ft. excess channel capacity, appears to be slight.

In case Lakes Michigan-Huron, under these circumstances, are at a stage where the addition of 0.4 ft. is undesirable, the excess volume of 50 000 sec-ft. may be simultaneously discharged through the St. Clair River to Erie, through the Niagara to Ontario, and through the St. Lawrence to the sea, without any level change in the intermediate lakes. The Superior Reservoir should be regarded as a unit with the Michigan-Huron Reservoir. When Michigan-Huron has a greater supply than it needs for its level maintenance within safe limits, the throttling of the flow of the St. Marys River will retain any desirable quantity in the Superior Reservoir. Conversely, when Michigan-Huron needs additional water, this may be secured by release from Superior. The immediate passing of water to the sea by opening simultaneously all works to the maximum extent of 50 000 sec-ft. will dispose of the excess supply.

Storage Reservoirs

The areas of the land part of the Superior Basin and of the reservoir are shown on Fig. 7, the Storage Reservoirs and Forebays of the Great Lakes System.

A depth of storage of 1 ft. on Superior has a value of 28 000 sec-ft. over a period of 1 year, and the combined storage capacity of the Superior and Michigan-Huron Reservoirs, for a pull-down of 1 ft., amounts to 68 000 sec-ft. over a period of 1 year, or 20 000 sec-ft. for a period of 3.4 years. It will be noted in Table 2 that the maximum net storage release from all the Great Lakes, including the forebays of Erie and Ontario, during the years 1921 to 1923, inclusive, amounts to about 24 000 sec-ft.; and that the maximum net release required from all reservoirs and forebays in the exceptional low-supply, 4-year

period, 1921-25, is 25 000 sec-ft. The storage value of the Superior and Michigan-Huron Reservoirs alone over a period of 5 years amounts to 13 600 sec-ft., while the capacity of all reservoirs and forebays for a depth of 1 ft. is nearly 17 000 sec-ft.

Michigan-Huron Supply

The outflow conditions in the St. Marys River are simple, and it is known that the average Superior supply amounts to about 80 000 sec-ft. It is known also (see Table 3) that the average supply outflowing in the Niagara River is 205 000 sec-ft. The average supply coming from Lakes Michigan-Huron and Erie is, therefore, 125 000 sec-ft. If it should be assumed arbitrarily that the Erie supply is 20 000 sec-ft., then Michigan-Huron shows an average supply of 105 000 sec-ft. The average outflow of the St. Clair River is then about 185 000 sec-ft. This outflow value is somewhat startling, because the outflow of the St. Clair, when Lake Huron is at its average stage of 581.1, is 205 000 sec-ft. There is, therefore, a discrepancy of 20 000 sec-ft. This is attributable to the fact that during the months of January to April, inclusive, the outflow of the St. Clair River under ice conditions is restricted to perhaps 150 000 sec-ft. In considering manual regulation of Michigan-Huron, the condition during the ice months mentioned must be given the fullest consideration.

St. Clair River Winter Flow

The small winter discharge capacity of the St. Clair River, suggested as perhaps 150 000 sec-ft., may doubtless be increased, if desirable, by channel improvements, more particularly in the delta section of the river. The hydraulic explanation of this small winter outflow is simple. In the open season the river utilizes 5.5 ft. of head as potential to meet the various energy expenditures in its 40 miles of channel. When winter comes, it finds itself flowing in part under an ice cover with diminished cross-sectional area. The various resistances may then be doubled, requiring a fall of 11.0 ft. instead of 5.5 ft. The result, therefore, with a fixed head of 5.5 ft., must inevitably be lessened volume of flow. Notwithstanding this lessened winter flow of the St. Clair River—somewhat augmented now by channel improvements—the Niagara River continues its outflow in ample volume, depending on Erie storage and local supply to supplement the lessened St. Clair River inflow; and the St. Lawrence River with its local supply, Ontario storage, and ice retardation, continues in ample flow.

Michigan-Huron Regulation

The suggested regulating works at the head of the St. Clair River for manual control, and the necessary vessel passes are shown in Fig. 5. The foundation for all works is boulder clay. The regulating works indicated, contemplate 3 000 ft. of 80-ft. sluices with massive Stoney gates, similar to those proposed by the Board of Engineers on Deep Waterways in 1900 for the head of the Niagara River. (See Plate VI.) Based on the estimate made by the Board of Engineers for the Niagara River, the cost of the regulating works

will be about $1 000 per lin. ft., or about $3 000 000. In addition will be the cost of lateral pier work for the east section, $500 000; and 1 700 ft. of sluice dam west of the expansion basin walls, costing perhaps $500 000. The total cost of regulating works is about $4 000 000. The three navigable passes and the two concrete expansion basins, will cost about $5 000 000. Breakwater protection—not shown in Fig. 5—above the uppermost pass will cost about $1 000 000. The total cost of all works at the head of the St. Clair River will be about $10 000 000. No provision is made for excess outflow capacity other than that due to the superelevation of the water surface of Lake Huron, and that due to enlarged navigable channels throughout the St. Clair River. No specific item of cost is assigned for dredge work within the expansion basin, and forming part of the large project of channel excavation required for 25 and 26-ft. navigation. As the vessel passes are no-stop works, no costs of operation similar to the costs at the ship locks at Sault Ste. Marie are involved. The ultimate capitalized value of these works to navigation and water power will be not less than $70 000 000.

The regulation of Michigan-Huron contemplates an elevation at the first of the year of 582.0 ft., with a cresting elevation of 583.0 ft. These elevations are subject to an emergency pull-down of 1 ft. for the use of storage for lake-level preservation in Lakes Erie and Ontario and the maintenance of uniform flow in the Niagara and St. Lawrence Rivers for water-power purposes.

The inertia of this vast reservoir safeguards riparians against high water. With the Michigan-Huron Reservoir at Elevation 582.2, at the beginning of April of a given year, it will require an excess supply from its own water-shed of 120 000 sec-ft., continuing for 4 months, to raise the lake to Elevation 583.2, and this is still 0.3 ft. below the cresting elevation in July, 1876, and June, 1886, and 1.5 ft. below the high water of 1838. Should an excess flow capacity in the St. Clair River of 50 000 sec-ft. be invoked to check this rise, the elevation in the July cresting in the illustrative case will be 582.8 ft. instead of 583.2 ft. The normal rise, with St. Marys River inflow without control, is 0.7 ft. between the first of April and the last of July. After July, the level of the Michigan-Huron Reservoir almost invariably drops.

The geographical regions embraced in the water-sheds of the Superior and Michigan-Huron Reservoirs are so extended that the diversity element of supply enters to a considerable extent. When the Superior Reservoir is receiving an exceptional supply, it is by no means certain that the Michigan-Huron Reservoir in its own basin is receiving a comparably large supply. The diversity element is more conspicuous still, in the regional differences between Superior and Erie and Ontario.

Restoration

On January 1, 1925, Lakes Michigan-Huron were 3½ ft. below the desirable regulated stage of 582.0 ft. To restore these lakes to this elevation will require a throttling of the outflow to the extent of 28 000 sec-ft. for a period of 5 years. The probability is small of level restoration without artificially throttling the flow of the St. Clair River. Reference to Table 4 indicates that the probability of a net excess volume of 20 000 sec-ft. for a

5-year period is 1 in 12. In the period, 1880-84, the net excess was at the rate of 19 000 sec-ft. The restoration of the Superior and Michigan-Huron Reservoirs, therefore, must be accomplished by lessened flow in the St. Clair, Niagara, and St. Lawrence Rivers over a period of from 5 to 10 years. Regulation in the Niagara River by works built simultaneously with regulating works above the head of the St. Clair River should precede the throttling of the outflow of Lake Huron for level restoration. This throttling should be accomplished in as large a measure as practicable during the non-navigation period between mid-December and mid-April.

Instantaneous Water Delivery

The infirmity of the natural operation of the Superior and Michigan-Huron Reservoirs is in the fact that it is impossible to deliver water to Lakes Erie and Ontario without the lapse of a long time interval. Should the St. Lawrence River need additional water at a time when Lake Superior is filled to the limit of its storage capacity, it would take 5 years to transmit this water in some such volume as 20 000 sec-ft. to the head of the St. Lawrence River. The Superior water released by the regulating works at Sault Ste. Marie must first raise the level of Lakes Michigan-Huron practically 1 ft. before the St. Clair River is able to discharge the 20 000 sec-ft. inflowing from St. Marys River, and Lake Erie likewise must rise nearly 1 ft. before it, in turn, will be able to transmit the water to Lake Ontario.

The outstanding value of co-ordinated regulation from the point of view of lake-level maintenance in Lakes Erie and Ontario and water-power use in the Niagara and St. Lawrence Rivers, arises from the fact that storage in the Superior Reservoir may be made immediately available, without a day's loss of time, for use in the St. Lawrence River. The procedure is as follows:

The St. Lawrence River needs 20 000 sec-ft. additional water. Instantaneously release 20 000 sec-ft. through the St. Marys River, the St. Clair River, the Niagara River, and the St. Lawrence River. The St. Lawrence then uses immediately the needed 20 000 sec-ft., without any change in the surface elevations of Lakes Michigan-Huron, Erie, or Ontario. This procedure is like cashing at Ogdensburg a draft on funds at Duluth. The ideal development at the St. Lawrence River in a series of four slack-water pools corresponds very closely with the geological layout of the four slack-water pools of the Great Lakes, and the transmission of water for peak-load use through the St. Lawrence from pool to pool will follow the instantaneous procedure described. Having in mind the fact that at present it takes a number of years to transmit available storage water from the Superior Reservoir to the head of the Niagara River, the tremendous economic gain in water use in the instantaneous transmission proposed by the writer is obvious.

At the Annual Convention of the Society in July, 1898, a paper by the late Hiram M. Chittenden, M. Am. Soc. C. E., then Captain, Corps of Engineers, U. S. A., entitled "Reservoir System of the Great Lakes of the St. Lawrence Basin",* with a "Mathematical Analysis of the Influence of Reser-

* *Transactions*, Am Soc. C. E., Vol. XL (1898), p. 355.

voirs Upon Stream-Flow", by the late James A. Seddon, M. Am. Soc. C. E., was presented by Mr. Seddon. In this paper the lag in time between the release of water from Lake Superior and its outflow in the St. Lawrence River was fully discussed.

Erie Regulating Works

The regulating works at the head of the Niagara River, with breakwater protection, as shown in Fig. 8, might cost approximately as follows: Sluices with Stoney gates, 4 000 lin. ft., $4 000 000; breakwaters, 3 miles in length, $3 000 000; channel enlargements for excess capacity flow, $3 000 000; total, $10 000 000. It will be understood that these estimates are roughly approximate. The Black Rock ship lock recommended by the Board of Engineers on Deep Waterways in 1900, to by-pass works at the head of the Niagara River, has already been built.

Expedients

Doubtless preliminary inexpensive measures may be used to restore, in part, the levels of Lakes Michigan-Huron. Submerged weirs in the St. Clair River and regulating works with gates in the Niagara River, to raise Lake Erie level and by back-water reflection the level of Lakes Michigan-Huron, will restore water-level losses chargeable to the diversion of 10 000 sec-ft. at Chicago. If no further betterment of lake levels or of water supply is needed, these simple expedients will serve. The future, however, will require the adequate regulation suggested by the writer in this paper. The permanent use of submerged weirs or wing dams violates the fundamental principle of regulation, which requires excess channel capacity in the St. Clair, Niagara, and St. Lawrence Rivers.

Canals as Excess Outlets

It should be understood that all continuously flowing canals—including the Drainage Canal at Chicago, the Black River Canal at Port Huron, the Welland Canal, and the Erie and Black Rock Canals at Buffalo—count as excess capacity channels required by the ultimate effective regulation of the Great Lakes System. To the extent of the volume of flow permissible in these various canals, the enlargement cost of the natural channels of the St. Clair and Niagara Rivers will be lessened. This is clearly the case in the St. Marys River, where the entire excess capacity flow needed, has already been secured in lateral water-power canals. The use of 10 000 sec-ft. of continuous flow in the Welland Canal for water power, if that be practicable, would also lessen the cost of rock excavation in the Niagara River. To this extent, in the ultimate analysis, it is obvious that excess-flow canals serve beneficially.

Data

In order that those discussing this paper may have available full information, the following diagrams are incorporated: Plate III is the hydrograph of the U. S. Lake Survey, showing the water surface elevations of the Great

Lakes for 65 years; Fig. 11 shows curves of discharge of the St. Clair River in terms of the elevation of Lake Huron and Lake St. Clair; Fig. 12 shows the storage capacity of Lake Ontario; Fig. 13 is the curve of discharge of the Niagara River in terms of the elevations of Lake Erie height at Buffalo; Fig. 14 gives the discharge curves on the St. Lawrence River before and after the building of Gut Dam at the Galops Rapids in 1903; Fig. 15 gives a series of curves expressing relationship between discharge in the St. Lawrence River and storage in Lake Ontario; Fig. 16 shows the theoretical division of discharge between the American and Canadian channels at the Galops Rapids of the St. Lawrence River; and Plate VII shows the detail of discharge measurements in the Niagara River, giving some indication of the precision of measurements.

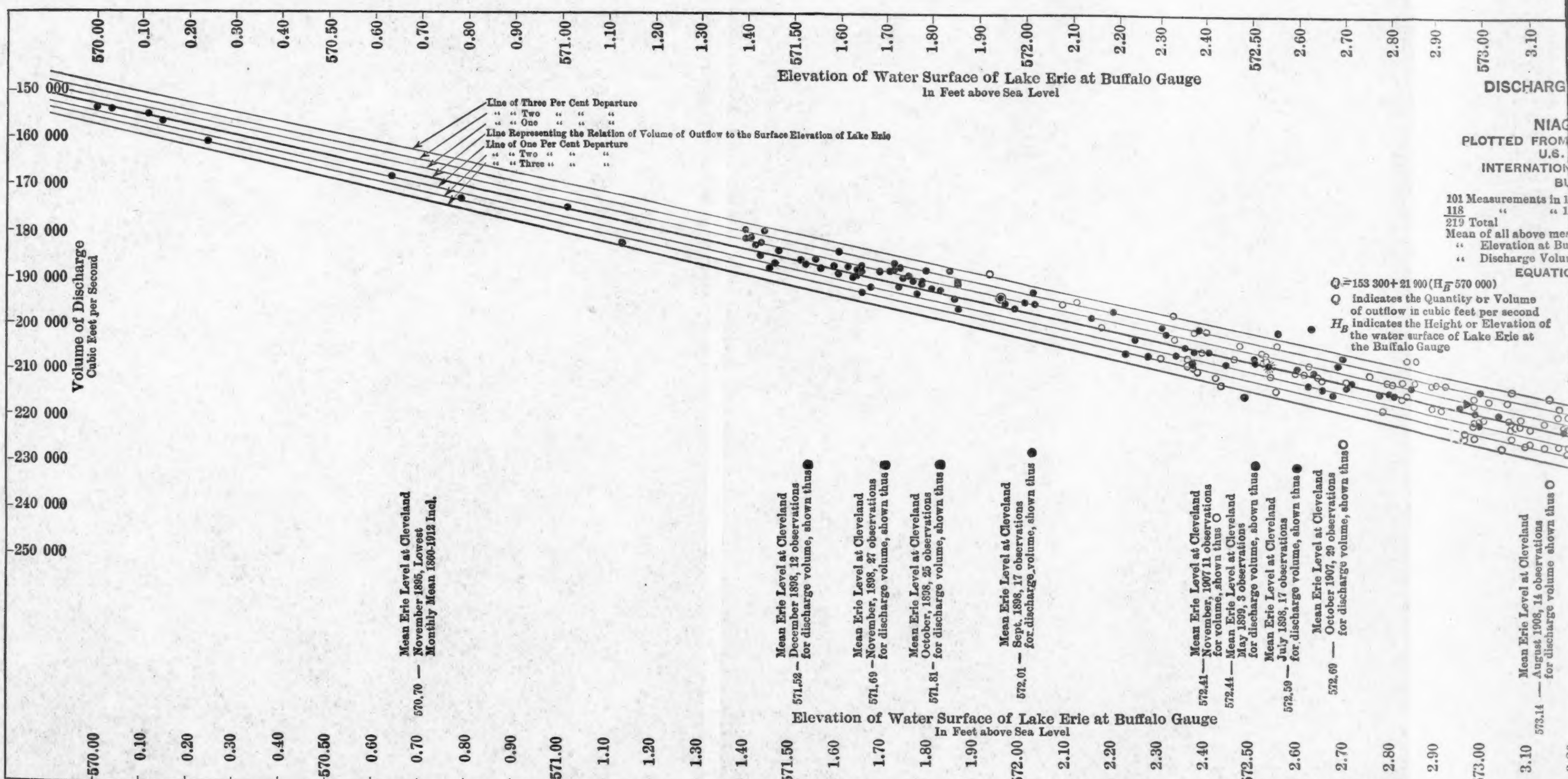
Financing the Work

When the time comes to adjust between the United States and Canada the cost of the various constructions of the St. Lawrence Waterway to the Sea, the full length from Duluth, Port Arthur, and Chicago, to the foot of the canalized sections of the St. Lawrence River below Montreal, must be taken into consideration.

Should a flow of 10 000 sec-ft. be utilized for water-power purposes in the Welland Canal, securing perhaps 250 000 e. h. p., rentals might be charged for the delivered water, and the Welland Canal would then be subsidized, in part, by water power. It is permissible under the Federal Water Power Act to assess money costs and charges for operation, maintenance, and depreciation of head-waters improvements against the water powers benefited, which might include those at Niagara Falls. The Act expressly states that a charge may be made by the United States for head-waters improvements to be paid by licensees. An administrative fee of 25 cents per h. p. is now paid by the Niagara Falls Power Company. This must amount to nearly $100 000 per year. On the Canadian side, rentals are charged for the use of the water, ranging from 50 cents to $1.00 per e. h. p.-year. The writer is not informed as to whether or not this includes all water power on the Canadian side at Niagara Falls. After the full development of 2 800 000 e. h. p. at Niagara Falls, an assessment of 10 cents per e. h. p.-year would bring in an annual return of $280 000, which capitalized at 5% is $5 600 000. Should St. Lawrence River water power ultimately pay a similar fee for head-waters improvements, the annual return there would be over $400 000 per year, with a capitalized value of $8 000 000. As navigation interests are large beneficiaries of the head-waters improvements suggested, the charge of 10 cents per e. h. p.-year against water power appears to be sufficient. The Sanitary District of Chicago has already offered to pay over $2 000 000 toward works for lake-level restoration.

International

The international aspect of the St. Lawrence Waterway to the Sea, with the various engineering works involved, needs careful consideration. Regulating works in St. Marys River, at the head of the St. Clair and Niagara and



Elevation of Water Surface of Lake Erie at Buffalo Gauge
In Feet above Sea Level
DISCHARG
NIAG
PLOTTED FROM
U.S.
INTERNATION
BU
101 Measurements in 1
118 " " 1
219 Total
Mean of all above mea
" Elevation at Bu
" Discharge Volu
EQUATIO
Line of Three Per Cent Departure
" " Two " " "
" " One " " "
Line Representing the Relation of Volume of Outflow to the Surface Elevation of Lake Erie
Line of One Per Cent Departure
" " Two " " "
" " Three " " "
Volume of Discharge
Cubic Feet per Second
150 000
160 000
170 000
180 000
190 000
200 000
210 000
220 000
230 000
240 000
250 000
Q = 153 300 + 21 900 (H_B - 570 000)
Q Indicates the Quantity or Volume of outflow in cubic feet per second
H_B indicates the Height or Elevation of the water surface of Lake Erie at the Buffalo Gauge
570.70 — Mean Erie Level at Cleveland November 1895, Lowest Monthly Mean 1860-1912 Incl.
571.52 — Mean Erie Level at Cleveland December 1898, 12 observations for discharge volume, shown thus ●
571.69 — Mean Erie Level at Cleveland November, 1898, 27 observations for discharge volume, shown thus ●
571.81 — Mean Erie Level at Cleveland October, 1898, 25 observations for discharge volume, shown thus ●
572.01 — Mean Erie Level at Cleveland Sept. 1898, 17 observations for discharge volume, shown thus ●
572.41 — Mean Erie Level at Cleveland November, 1907 11 observations for volume, shown thus ○
572.44 — Mean Erie Level at Cleveland May 1899, 3 observations for discharge volume, shown thus ●
572.59 — Mean Erie Level at Cleveland July 1898, 17 observations for discharge volume, shown thus ●
572.69 — Mean Erie Level at Cleveland October 1907, 29 observations for discharge volume, shown thus ○
573.14 — Mean Erie Level at Cleveland August 1908, 14 observations for discharge volume shown thus ○

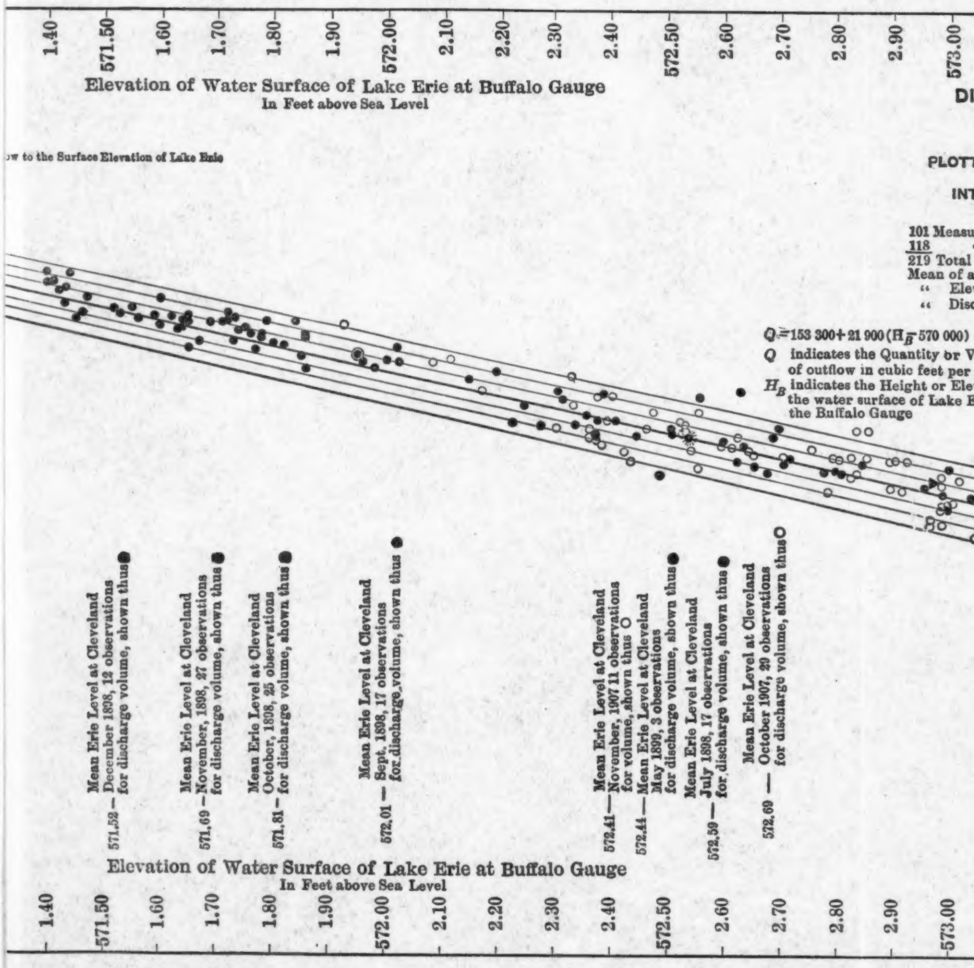
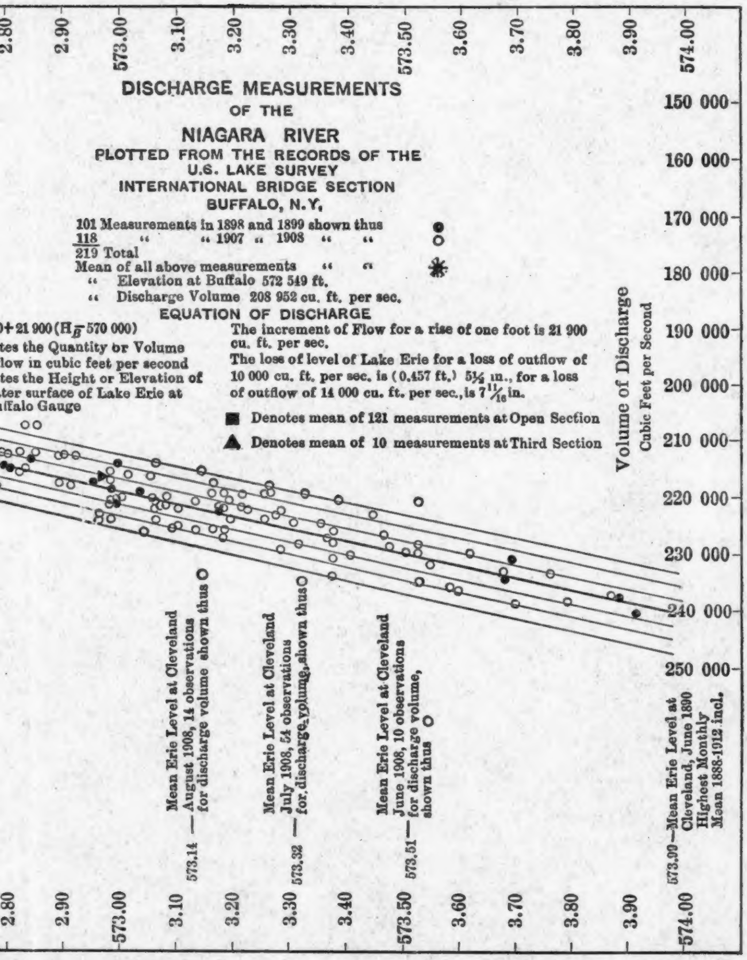

1.40
571.50
1.60
1.70
1.80
1.90
572.00
2.10
2.20
2.30
2.40
572.50
2.60
2.70
2.80
2.90
573.00
Elevation of Water Surface of Lake Erie at Buffalo Gauge
In Feet above Sea Level
w to the Surface Elevation of Lake Erie
PLOTT
INT
101 Measu
118
219 Total
Mean of a
" Ele
" Disc
Q=153 300+21 900(H_B-570 000)
Q indicates the Quantity or V
of outflow in cubic feet per
H_B indicates the Height or Ele
the water surface of Lake E
the Buffalo Gauge
571.52 — Mean Erie Level at Cleveland
December 1898, 12 observations
for discharge volume, shown thus
571.69 — Mean Erie Level at Cleveland
November, 1898, 27 observations
for discharge volume, shown thus
571.81 — Mean Erie Level at Cleveland
October, 1898, 25 observations
for discharge volume, shown thus
572.01 — Mean Erie Level at Cleveland
Sept. 1898, 17 observations
for discharge volume, shown thus
572.41 — Mean Erie Level at Cleveland
November, 1907 11 observations
for volume, shown thus O
572.44 — Mean Erie Level at Cleveland
May 1899, 3 observations
for discharge volume, shown thus
572.59 — Mean Erie Level at Cleveland
July 1898, 17 observations
for discharge volume, shown thus
572.69 — Mean Erie Level at Cleveland
October 1907, 20 observations
for discharge volume, shown thus O
Elevation of Water Surface of Lake Erie at Buffalo Gauge
In Feet above Sea Level
1.40
571.50
1.60
1.70
1.80
1.90
572.00
2.10
2.20
2.30
2.40
572.50
2.60
2.70
2.80
2.90
573.00

PLATE VII.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1925
SHENEHON ON
THE ST. LAWRENCE WATERWAY
TO THE SEA



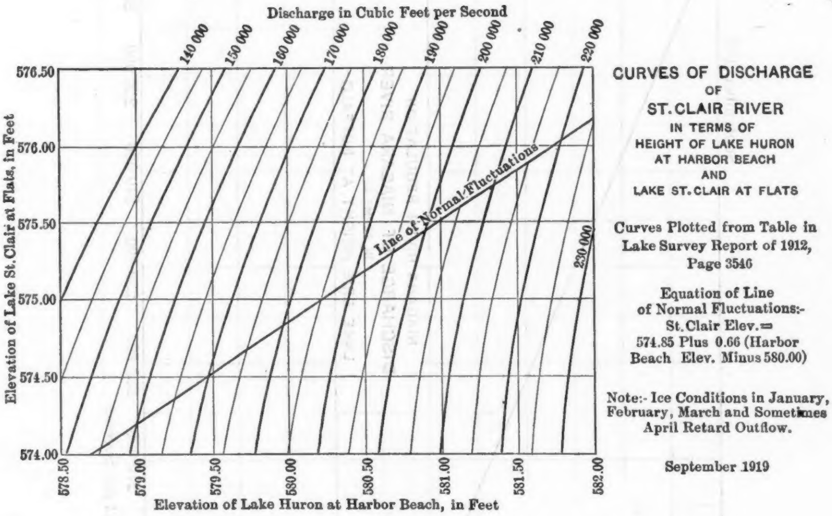

Discharge in Cubic Feet per Second
140 000
150 000
160 000
170 000
180 000
190 000
200 000
210 000
220 000
230 000
Line of Normal Fluctuations
Elevation of Lake St. Clair at Flats, in Feet
576.50
576.00
575.50
575.00
574.50
574.00
578.50
579.00
579.50
580.00
580.50
581.00
581.50
582.00
Elevation of Lake Huron at Harbor Beach, in Feet
CURVES OF DISCHARGE
OF
ST. CLAIR RIVER
IN TERMS OF
HEIGHT OF LAKE HURON
AT HARBOR BEACH
AND
LAKE ST. CLAIR AT FLATS
Curves Plotted from Table in
Lake Survey Report of 1912,
Page 3546
Equation of Line
of Normal Fluctuations:-
St. Clair Elev. =
574.85 Plus 0.66 (Harbor
Beach Elev. Minus 580.00)
Note:- Ice Conditions in January,
February, March and Sometimes
April Retard Outflow.
September 1919

FIG. 11.

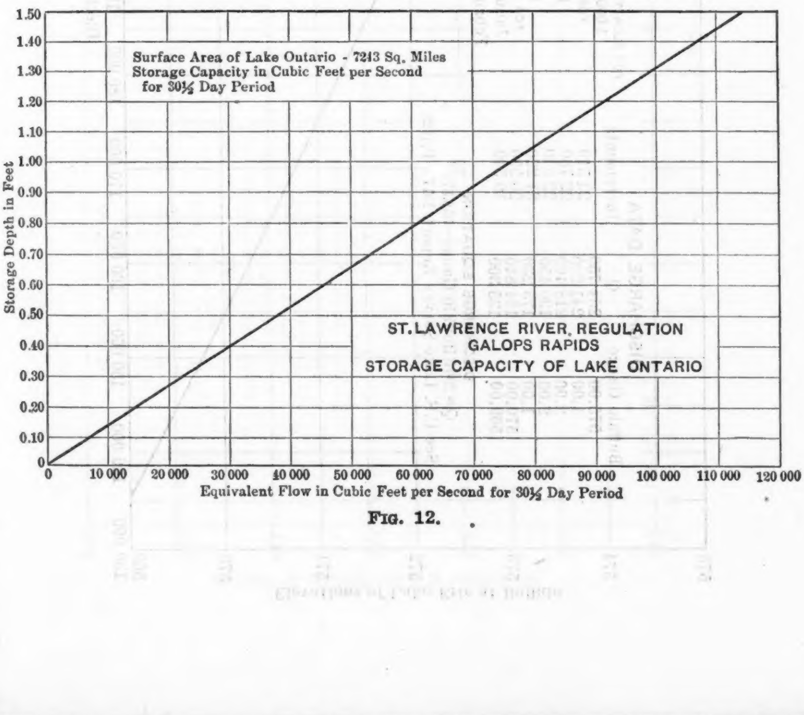

Surface Area of Lake Ontario - 7243 Sq. Miles
Storage Capacity in Cubic Feet per Second
for 30½ Day Period
ST. LAWRENCE RIVER, REGULATION
GALOPS RAPIDS
STORAGE CAPACITY OF LAKE ONTARIO
Storage Depth in Feet
1.50
1.40
1.30
1.20
1.10
1.00
0.90
0.80
0.70
0.60
0.50
0.40
0.30
0.20
0.10
0
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
110 000
120 000
Equivalent Flow in Cubic Feet per Second for 30½ Day Period

FIG. 12.

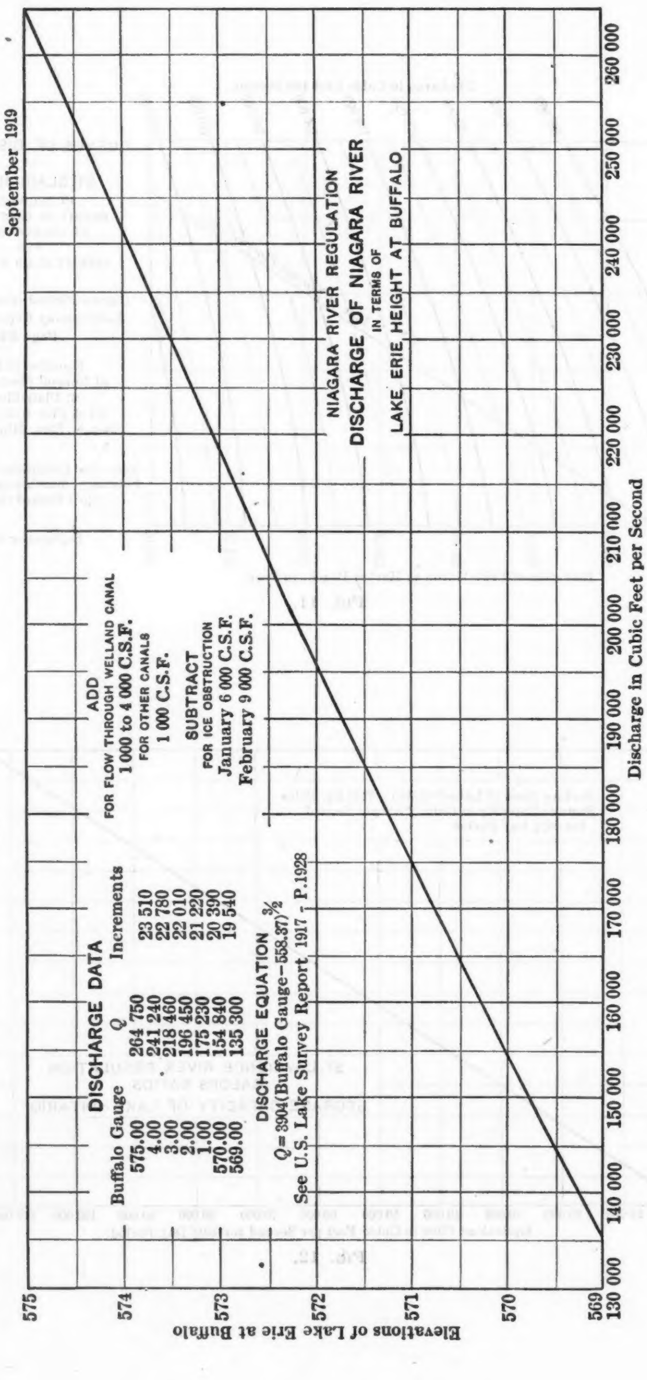

September 1919
NIAGARA RIVER REGULATION
DISCHARGE OF NIAGARA RIVER
IN TERMS OF
LAKE ERIE HEIGHT AT BUFFALO
ADD
FOR FLOW THROUGH WELLAND CANAL
1 000 to 4 000 C.S.F.
FOR OTHER CANALS
1 000 C.S.F.
SUBTRACT
FOR ICE OBSTRUCTION
January 6 000 C.S.F.
February 9 000 C.S.F.
DISCHARGE DATA
Buffalo Gauge Q Increments
575.00 264 750 23 510
4.00 241 240 22 780
3.00 218 460 22 010
2.00 196 450 21 220
1.00 175 230 20 390
570.00 154 840 19 540
569.00 135 300
DISCHARGE EQUATION
Q = 3904(Buffalo Gauge − 553.37)^3/2
See U.S. Lake Survey Report 1917 - P.1928
575
574
573
572
571
570
569
Elevations of Lake Erie at Buffalo
130 000
140 000
150 000
160 000
170 000
180 000
190 000
200 000
210 000
220 000
230 000
240 000
250 000
260 000
Discharge in Cubic Feet per Second

FIG. 13.

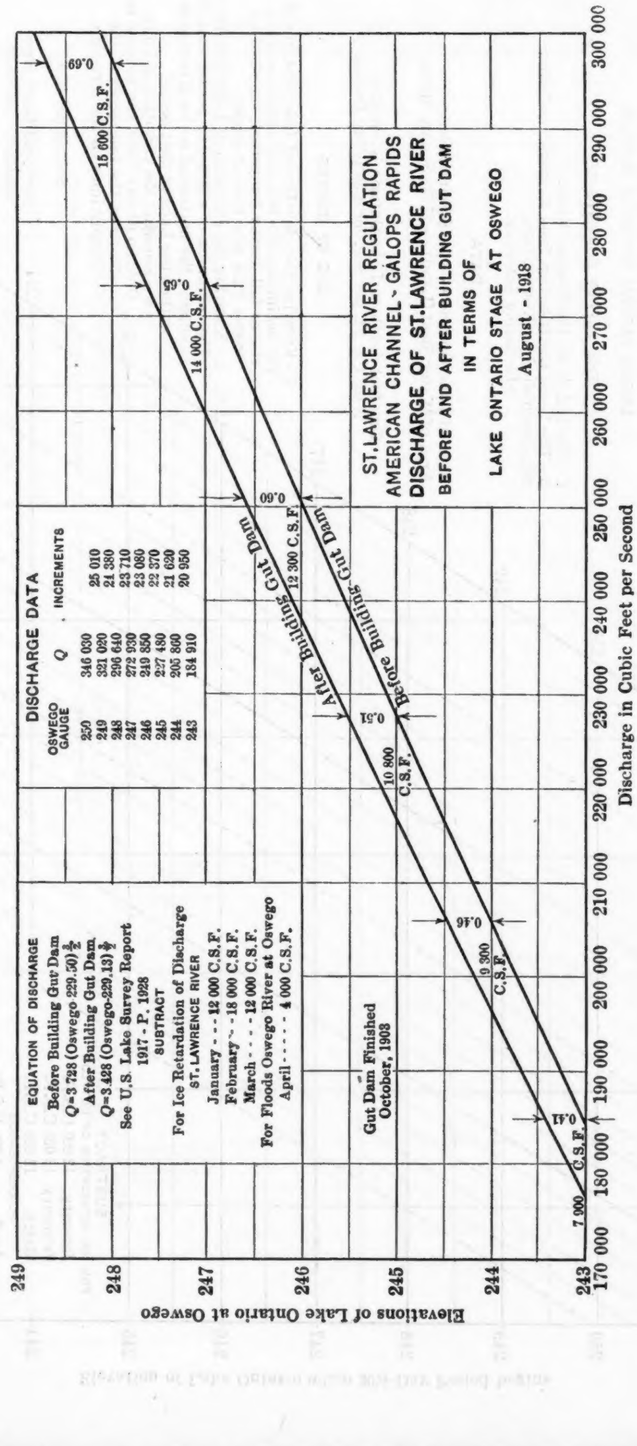

ST. LAWRENCE RIVER REGULATION
AMERICAN CHANNEL - GALOPS RAPIDS
DISCHARGE OF ST. LAWRENCE RIVER
BEFORE AND AFTER BUILDING GUT DAM
IN TERMS OF
LAKE ONTARIO STAGE AT OSWEGO
August - 1918
EQUATION OF DISCHARGE
Before Building Gut Dam
Q=3 728 (Oswego-229.50)$\frac{3}{2}$
After Building Gut Dam
Q=3 428 (Oswego-229.13)$\frac{3}{2}$
See U.S. Lake Survey Report
1917 - P. 1928
SUBTRACT
For Ice Retardation of Discharge
ST. LAWRENCE RIVER
January - - - 12 000 C.S.F.
February - - 18 000 C.S.F.
March - - - - 12 000 C.S.F.
For Floods Oswego River at Oswego
April - - - - 4 000 C.S.F.
DISCHARGE DATA
OSWEGO GAUGE | Q | INCREMENTS
250 | 346 030 |
249 | 321 020 | 25 010
248 | 296 640 | 21 380
247 | 272 930 | 23 710
246 | 249 850 | 23 080
245 | 227 480 | 22 370
244 | 205 860 | 21 620
243 | 184 910 | 20 950
Gut Dam Finished
October, 1903
After Building Gut Dam
Before Building Gut Dam
7 900 C.S.F.
0.41
9 300 C.S.F.
0.46
10 800 C.S.F.
0.51
12 300 C.S.F.
0.60
14 000 C.S.F.
0.66
15 600 C.S.F.
0.69
249
248
247
246
245
244
243
Elevations of Lake Ontario at Oswego
170 000
180 000
190 000
200 000
210 000
220 000
230 000
240 000
250 000
260 000
270 000
280 000
290 000
300 000
Discharge in Cubic Feet per Second

FIG. 14.

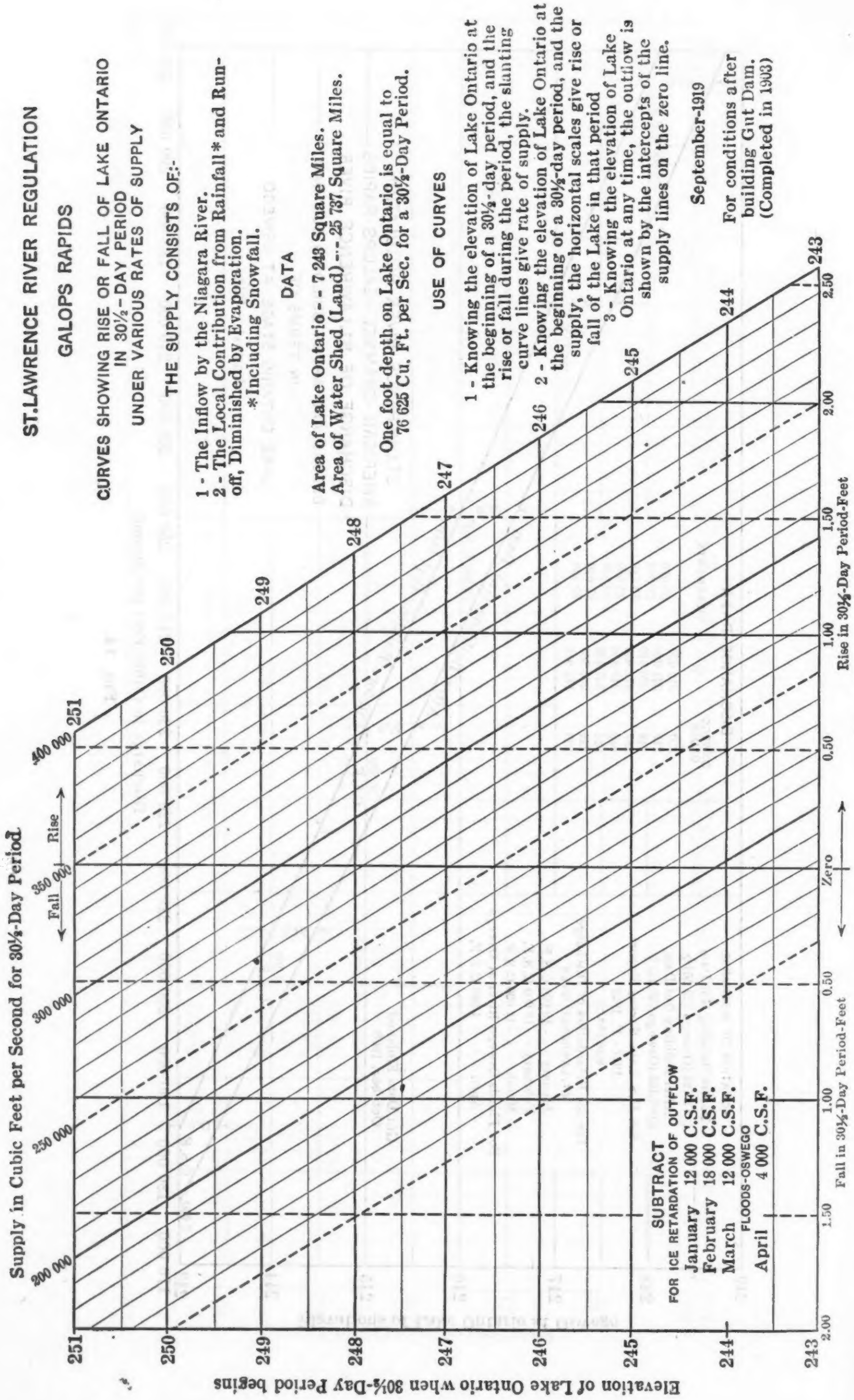

ST. LAWRENCE RIVER REGULATION
GALOPS RAPIDS
CURVES SHOWING RISE OR FALL OF LAKE ONTARIO
IN 30½-DAY PERIOD
UNDER VARIOUS RATES OF SUPPLY
THE SUPPLY CONSISTS OF:-
1 - The Inflow by the Niagara River.
2 - The Local Contribution from Rainfall* and Run-off, Diminished by Evaporation.
*Including Snowfall.
DATA
Area of Lake Ontario - - 7 243 Square Miles.
Area of Water Shed (Land) - - 25 737 Square Miles.
One foot depth on Lake Ontario is equal to 76 625 Cu. Ft. per Sec. for a 30½-Day Period.
USE OF CURVES
1 - Knowing the elevation of Lake Ontario at the beginning of a 30½-day period, and the rise or fall during the period, the slanting curve lines give rate of supply.
2 - Knowing the elevation of Lake Ontario at the beginning of a 30½-day period, and the supply, the horizontal scales give rise or fall of the Lake in that period
3 - Knowing the elevation of Lake Ontario at any time, the outflow is shown by the intercepts of the supply lines on the zero line.
September-1919
For conditions after building Gut Dam. (Completed in 1903)
Supply in Cubic Feet per Second for 30½-Day Period
200 000
250 000
300 000
350 000
400 000
Fall
Rise
251
250
249
248
247
246
245
244
243
Elevation of Lake Ontario when 30½-Day Period begins
SUBTRACT
FOR ICE RETARDATION OF OUTFLOW
January 12 000 C.S.F.
February 18 000 C.S.F.
March 12 000 C.S.F.
FLOODS-OSWEGO
April 4 000 C.S.F.
2.00
1.50
1.00
0.50
Zero
0.50
1.00
1.50
2.00
2.50
Fall in 30½-Day Period-Feet
Rise in 30½-Day Period-Feet

FIG. 15.

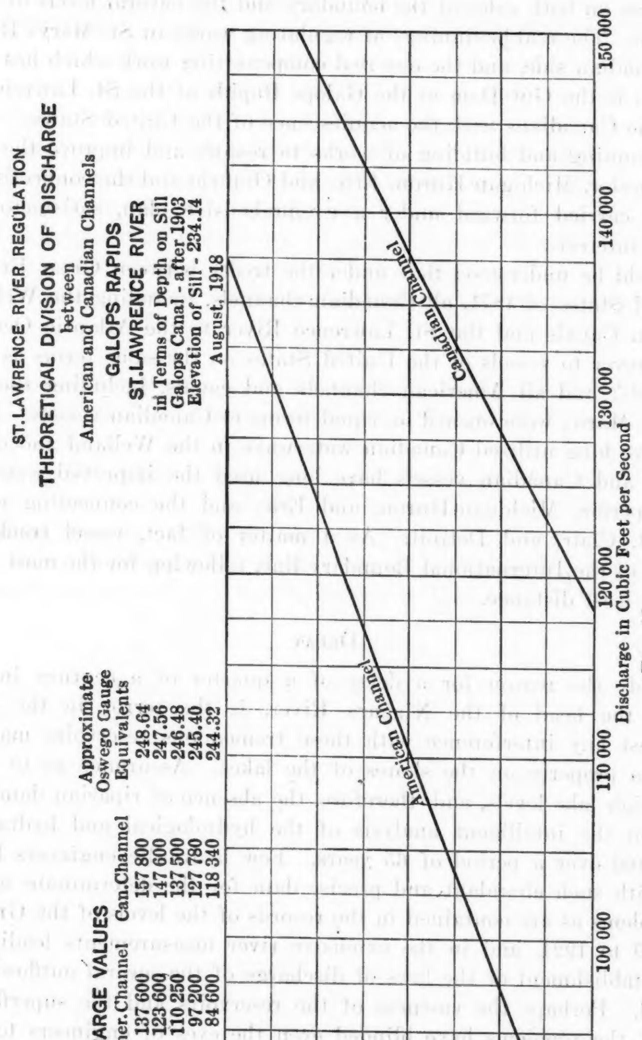

ST. LAWRENCE RIVER REGULATION
THEORETICAL DIVISION OF DISCHARGE
between
American and Canadian Channels
GALOPS RAPIDS
ST. LAWRENCE RIVER
in Terms of Depth on Sill
Galops Canal - after 1903
Elevation of Sill - 234.14
August, 1918
DISCHARGE VALUES
Depth on Sill Amr. Channel Can. Channel Approximate Oswego Gauge Equivalents
12.00 137 000 137 800 248.64
11.00 123 600 137 600 247.56
10.00 110 250 137 500 246.48
9.00 97 200 127 780 245.40
8.00 84 600 118 340 244.32
Canadian Channel
American Channel
Depth on Sill of Lock 27
12
11
10
9
8
85 000
90 000
100 000
110 000
120 000
130 000
140 000
150 000
Discharge in Cubic Feet per Second

Fig. 16.

in the St. Lawrence, bestride the boundary line and modify the natural outflow of the rivers on both sides of the boundary and the natural levels of the lakes above them. The real beginnings of regulating works in St. Marys River were on the Canadian side, and the one real compensating work which has benefited lake levels, is the Gut Dam at the Galops Rapids of the St. Lawrence River, built by the Canadians with the acquiescence of the United States.

The planning and building of works to restore and improve the levels of Lakes Superior, Michigan-Huron, Erie, and Ontario and the connecting rivers, should be carried forward under a comprehensive plan, satisfactory to all parties at interest.

It should be understood that under the treaty between Great Britain and the United States of 1871, all Canadian channels, including the Welland and Laurentian Canals and the St. Lawrence River to the Atlantic Ocean, were opened forever to vessels of the United States on the same terms as to Canadian vessels; and all American channels and canals, including the locks at Sault Ste. Marie, were opened on equal terms to Canadian vessels. American vessels have long utilized Canadian waterways in the Welland and on the St. Lawrence and Canadian vessels have long used the improved waterways of Lakes Superior, Michigan-Huron, and Erie, and the connecting rivers, St. Marys, St. Clair, and Detroit. As a matter of fact, vessel tracks are no respecters of the International Boundary line, following for the most part deep water and least distance.

Delay

Probably the reason for a delay of a quarter of a century in building works at the head of the Niagara River, is the terror in the minds of laymen lest any interference with these tremendous reservoirs may lead to disaster to property on the shores of the lakes. Assurance as to the exact limit of high lake levels, and, therefore, the absence of riparian damage, must come from the intelligent analysis of the hydrological and hydraulic data accumulated over a period of 65 years. Few hydraulic engineers have been blessed with such abundant and precise data for the determinate solution of their problems as are contained in the records of the levels of the Great Lakes from 1860 to 1924, and in the extensive river measurements leading to the precise establishment of the laws of discharge of the several outflowing rivers concerned. Perhaps the vastness of the reservoirs and the superficial complexity of the problems have blinded even the eyes of engineers to the simplicity, certainty, and desirability of the changes suggested in this paper.

New Boundary Waters Treaty

The present Boundary Waters Treaty of 1910 between Great Britain and the United States needs to be fundamentally rewritten; and when a new treaty is made, the counsel of hydraulic engineers will be needed. It should provide for the construction of the various works suggested in this paper, and it should insure the most effective use of all regulating works, including those in the St. Marys River, for the dual purpose of navigation and water power. Diversions should contemplate such elasticity as will permit the best economic use of the

water, with intermittent flow for peak-load conditions and augmented flow, if desirable, for power purposes during the winter season.

Such new treaty should, in general, determine and adjust all the various and interrelated matters which have to do with the entire question of the passing of ships from the western end of Lake Superior along all possible routes eastward to the Atlantic Ocean. The policy to be determined is one in which two nations are vitally interested and a new treaty must be broad and just. No real progress can be made and no substantial advantage can be secured except through negotiations, based squarely on equality of opportunity. Considerations of statistics, of expediency, of benefits, or of strategy of position must not be permitted to delay the solution of this great problem which looks to the economic welfare of the people of two countries.

Conclusion

This paper is of necessity long. It attempts to deal comprehensively with great projects for navigation and water power. It is intended to be constructive, provoking discussion to the end that a clearer comprehension of the engineering problems of the Great Lakes System may exist in the minds of engineers and of laymen, and that the best solutions may be devised. It is not intended to prescribe finalities, but rather to suggest the vital principles which should obtain in the engineering ways of securing ultimate benefits in this tremendous international resource of the St. Lawrence Waterway to the Sea.

MULTIPLE-ARCH DAM AT GEM LAKE ON RUSH CREEK, CALIFORNIA

By Fred O. Dolson* and Walter L. Huber,† Members, Am. Soc. C. E.

To Be Presented October 7, 1925.

Synopsis

This paper is submitted for the purpose of calling the attention of engineers to the necessity of giving serious consideration to the action of frost when designing dams for use in cold climates. The writers believe deterioration from frost action will occur in any thin concrete structure subjected to water pressure and extreme cold unless the concrete can be made 100% water-proof (or water-tight).

The results of frost action, therefore, have been pointed out in this paper, together with the expensive repairs necessary to safeguard the structure, and it is hoped the discussion will develop some positive and permanent method of water-proofing concrete.

Introduction

Much has been written in recent years concerning multiple-arch dams, but the writers believe that all the story is not yet told, and it is with the hope of placing before the Engineering Profession another valuable chapter that this contribution is offered. Much of the material heretofore presented has dealt wholly, or almost entirely, with mathematical analyses. It is not intended to attempt an addition to the store of material available along these lines. Indeed, a mathematical analysis and a complete description of the structure which is the subject of this paper has already appeared in the publications of the Society.‡

Gem Dam was built in 1915 and 1916, and at the time embodied all the best practice in the design of multiple-arch dams, as is evidenced by its designer, L. R. Jorgensen, M. Am. Soc. C. E., having been awarded the Norman Medal by the Society in 1917 for his paper entitled, "Multiple-Arch Dams on Rush Creek, California".‡ Eight years of subsequent operation of this structure, during which time the concrete has partly disintegrated by freezing, has afforded opportunities for observations and has necessitated remedial measures which, it is believed, cannot fail to be of interest.

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on this paper will be closed with the January, 1926, *Proceedings*. When finally closed, the paper, with discussion in full, will be published in *Transactions*.

* Vice-Pres. and Asst. Gen. Mgr., The Nevada California Power Co., Riverside, Calif.

† Cons. Civ. Engr., San Francisco, Calif.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

Descriptions of engineering structures which have failed to function as intended are often very instructive. It is hoped this one will prove to be as interesting to those members of the profession who are concerned with similar structures as it has been to the several engineers who have had to do with remedial methods.

Selection of Multiple-Arch Type

Gem Dam, on the head-waters of Rush Creek in Mono County, California, is well up on the eastern slope of the Sierra Nevada Mountains at an elevation of 9 050 ft. (water-surface level) and in one of the most remote localities in the West. All materials necessary for its construction except lumber, which could be cut locally, and rock, must be moved long distances and under difficult conditions. Cement had to be shipped from the place of manufacture by broad-gauge railroad 336 miles, transhipped to a narrow-gauge railroad, and hauled 84 miles farther; then hauled over a sandy desert road, using engines or motor trucks of the caterpillar type, for 70 miles, to the power-house below the dam. Here, it was reloaded on tram cars and raised more than 1 250 ft. vertically on a 4 826-ft. tramway to Agnew Lake, where it was rehandled, loaded on barges, and towed across the lake to be again rehandled and raised an additional height of 550 ft. by another tramway, being finally placed on the dam site at a total cost, even under 1915 conditions, of $7.50 per bbl. In the high altitude of this site, the season during which construction work can be advantageously conducted is necessarily short, often only four months and seldom longer than five months.

The dam site is one of exposed, hard, blue limestone and igneous granite, still showing in many places the polish placed on it in the Glacial Age by the glacier which formerly extended down from the higher Sierra. No earth and very little sand is found in this locality.

Physical conditions governing the original construction of Gem Dam had much to do with the selection of the multiple-arch type. Materials for an earth dam were not available. A masonry dam was thought, at the time, to have certain advantages over a rock-fill type. Because of the excessive cost of materials, the quantities required for a masonry dam of the gravity type had to be avoided. These considerations led to the selection of the multiple-arch type.

Description

Gem Dam was constructed in 1915 and 1916 at the outlet of Lower Gem Lake. The reservoir created by it has a storage capacity of 17 216 acre-ft., and is supplied by the run-off from a water-shed of 21.3 sq. miles, ranging in elevation from 9 000 ft. at the dam to 13 090 ft. at Mt. Lyell. The dam is composed of sixteen complete arches, of 40-ft. span, between centers of buttresses, and two fractional arches at the ends. The length at the crest is 688 ft. The maximum height of any individual arch is 84 ft., and the vertical distance from the crest to the deepest point in the foundation is 112 ft.

The arches vary in thickness from 1.0 ft. at the crest to 3.95 ft. at the deepest point, and the up-stream face is inclined at an angle of 50° from the horizontal. The buttresses vary in thickness from 1.85 ft. at their tops to 4.25 ft. at the deepest point, and are strengthened by counterforts, varying in total width from 4.5 ft. at their tops, which are 15 ft. below the crest of the dam, to 11.0 ft. at the deepest point. Two sets of double, horizontal, braced 12 by 30-in. struts extend between the buttresses, one set 15 ft. and one set 45 ft. below the crest. Spillway openings, which can be fitted with stop planks, have been left near the crest of the dam in the two southern arches, the five openings, 6 ft. long by 2 ft. deep, in the end arch being at Elevation 9 050, and the eight openings, 6 ft. long by 2 ft. deep, in the next arch at Elevation 9 048. A steel pipe line leads directly from the dam to the power-house, where a static head of 1 807 ft. is obtained.

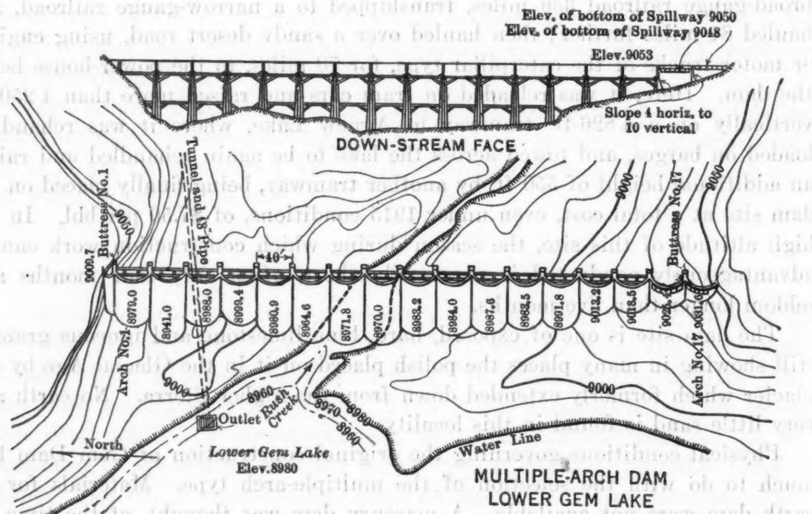


FIG. 1.

This general description, together with the plans, Figs. 1, 2, and 3, and the photographic views, Figs. 4, 5, and 6, should give the reader a fair idea of the structure as it existed from its completion in 1916 until its repair in 1924. A more complete description, including details of reinforcement and an analysis of stresses, is given in Mr. Jorgensen's paper, previously mentioned. A review of this analysis will show that the stresses are within the limits prescribed by good engineering practice and that the design is more conservative than that of other multiple-arch dams which have been constructed to greater heights.

HISTORY OF OPERATION

During the eight years of operation of the structure, very few cracks occurred in it. A horizontal crack developed in each of the arches immediately below its crest, due undoubtedly to the fact that the deflection of the walk

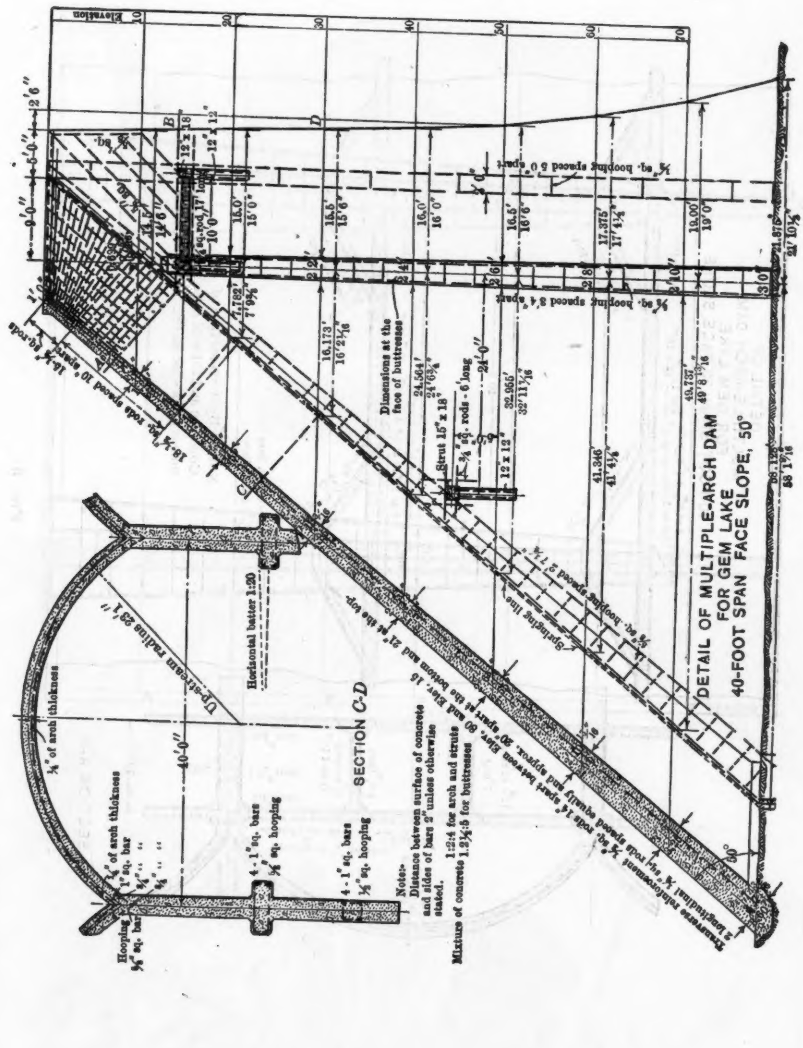

DETAIL OF MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN FACE SLOPE, 50°
SECTION C-D
Elevation
Dimensions at the face of buttresses
Horizontal batter 1:20
Up-stream radius 23½″
40′-0″
Springing line
Strut 15″ x 18″
12″ x 12″
Note:-
Distance between surface of concrete and sides of bars 2″ unless otherwise stated.
Mixture of concrete 1:2:4 for arch and struts
1:2½:5 for buttresses

FIG. 2.

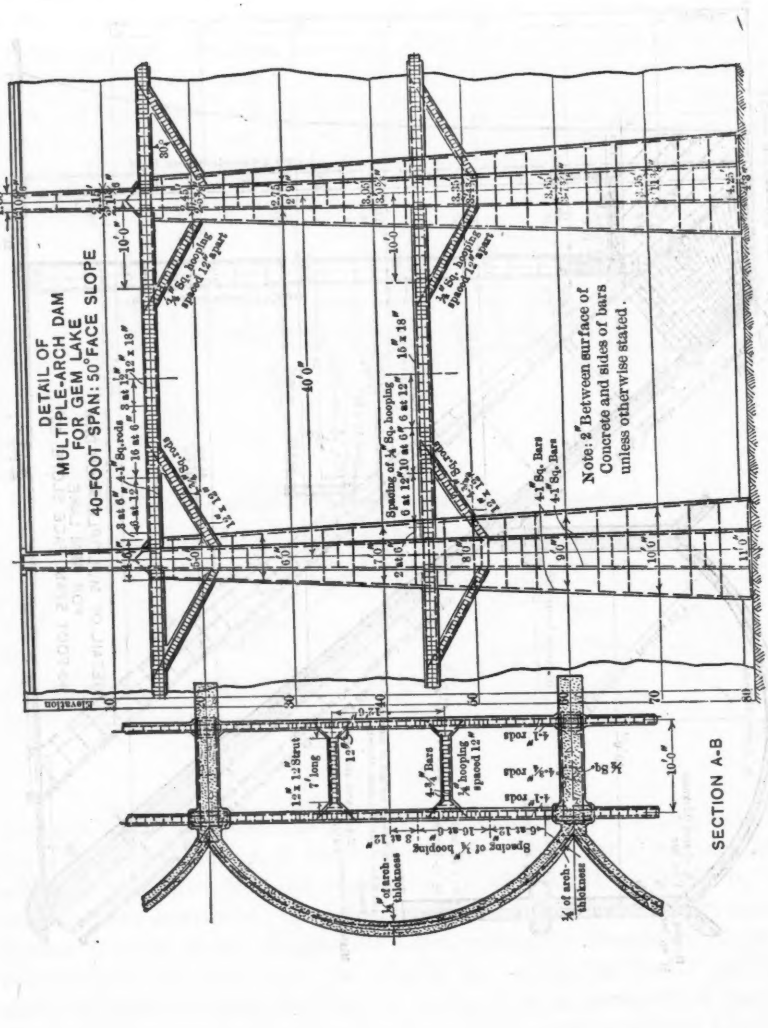

DETAIL OF
MULTIPLE-ARCH DAM
FOR GEM LAKE
40-FOOT SPAN: 50° FACE SLOPE
Note: 2" Between surface of
Concrete and sides of bars
unless otherwise stated.
SECTION A-B
Elevation

FIG. 3.

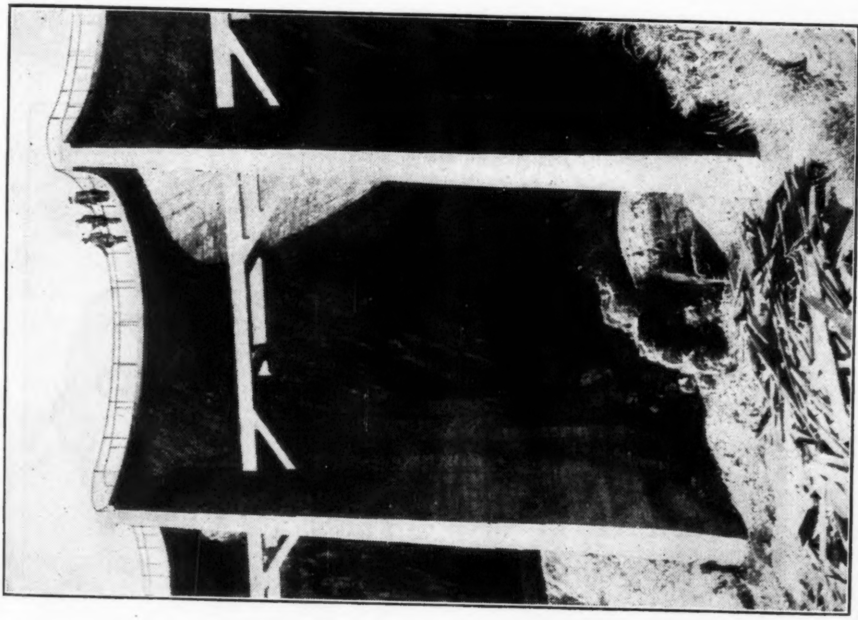


FIG. 5.—DETAILED VIEW OF REAR FACE OF ARCH No. 3, GEM DAM.

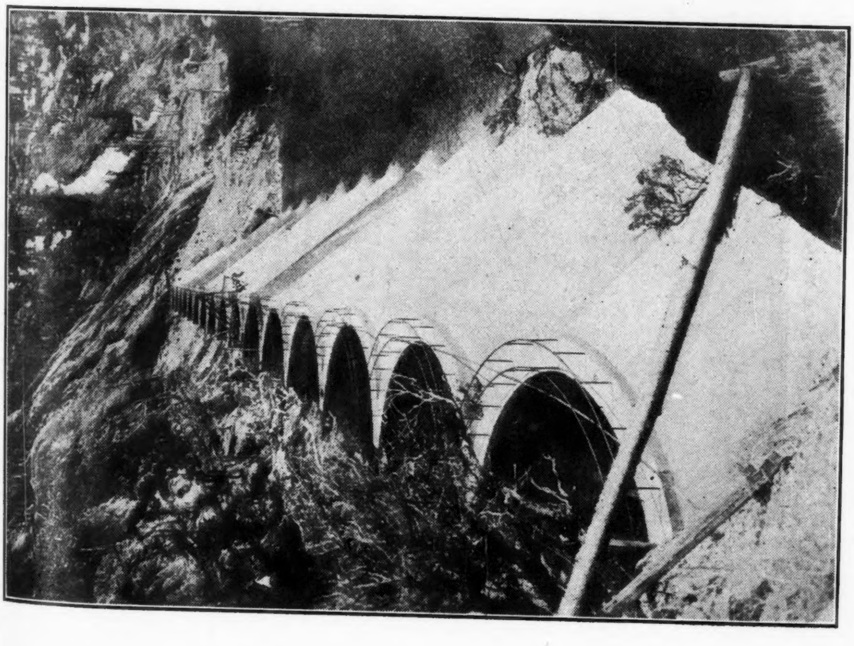


FIG. 4.—GENERAL VIEW OF GEM DAM.

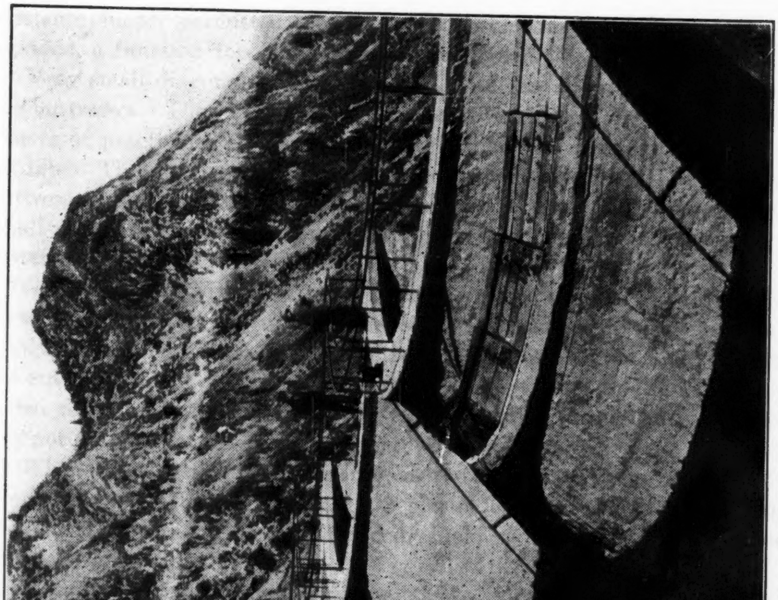


FIG. 6.—WATER FACE OF GEM DAM AFTER COMPLETION OF GUNITE COAT. NOTE OPERATION OF SPILLWAY WITH STOP PLANKS IN PLACE.

Pape

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along the crest, which acted as an arch between the tops of the buttresses, was less than the deflection of the thinner arches of the dam immediately below. This crack introduced no element of weakness and, because of its location and its very slight width, was not evident except to the critical observer. Its existence simply permitted the arches of the dam to resist all water pressure unaided, a function for which they were designed.

Very small diagonal cracks developed near the up-stream toes of three of the buttresses. They were similar to cracks which have developed in the buttresses of practically all the multiple-arch dams which have been constructed to date. The usual practice has been to provide but little reinforcement in buttresses or to omit it. It is believed that the introduction of a comparatively small amount of reinforcement would have eliminated these cracks in every instance.

Gem Dam has withstood some rather severe tests. In September, 1918, a very heavy rain on the upper water-shed occurred when the spillways were partly blocked with flash-boards, and before the flash-boards could be removed, the entire structure had been overtopped to a depth of 4 in. This overtopping lasted for 8 hours, but not to the maximum depth throughout. Although it was not designed as an overflow dam, no damage resulted.

When the structure was constructed, the up-stream faces of the arches of Gem Dam were covered with a 1 : 2 plaster coat of cement mortar, applied with a cement gun. This coat varied in thickness from ½ in. at the top to ¾ in. at the bottom.

At first, the leakage was very small. Mr. Jorgensen* thus describes it:

"Some of the arches on the Gem Lake Dam are absolutely tight, some of them sweat in places, and a few drip in places. A few small springs have formed behind the dam, and a trickle of water comes under one arch; but, all told, the total leakage is very small."

It remained small for two years after the completion of the dam and the filling of the reservoir, and during this time no change in the structure was apparent. During the third year some leakage developed and a white deposit was noticeable on the rear faces of the arches. At first, this condition was not considered serious, as a small amount of leakage through a concrete structure is not necessarily a cause for alarm. After chemical analyses, it was concluded that the white deposit was laitance which was being washed out of construction joints.

During the next two years serious disintegration started on the face of the dam, apparently due to the concrete freezing in thin layers on the face and disintegrating by frost action. This action which was slow at first, increased very rapidly with each additional year of age. It was originally believed that the damage had resulted from ice which formed on the surface of the water in the reservoir to a thickness of 30 or 40 in. This ice adhered to the surface of the dam and, as the water was lowered and the ice thus left overhanging, it would tear loose, presumably taking the gunite coating with it. However, this theory was partly disproved by cutting a hole through the ice near the

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 906.

dam and examining the water face immediately below. It was found that, for 8 or 10 ft. under the reservoir surface, this face was covered with a thin layer of ice, thus proving that the freezing was through the thin concrete section of the dam and that the ice on the surface of the reservoir was not the entire cause of the damage.

It was believed that the condition could be remedied if the water face of the structure could be effectively and permanently water-proofed. A careful and detailed study of possible methods of accomplishing this result was made. This study included laboratory tests by J. Y. Jewett, Assoc. M. Am. Soc. C. E., of twelve brands of water-proofing compounds, all of which failed to meet the requirements of the tests.*

The study of water-proofing compounds led to the selection of the so-called "Ironite" treatment. Before applying this treatment, all concrete that showed any signs of disintegration was chipped from the face of the dam. This proved to be quite a task. In places, the concrete over large areas was chipped to a depth of as much as 8 in. Concrete was very carefully replaced in these areas to insure a good bond, and the ends or corners of such areas were chipped out square in an attempt to preserve arch action. After the application of the "Ironite" treatment, the dam was practically water-tight for a short time. However, the "Ironite" coating soon developed hair-line cracks, probably due to expansion and contraction of the arches, and thereafter, with the return of low temperatures, deterioration began again. It was then apparent that more heroic remedial measures must be adopted.

Condition of Dam in 1924

A careful examination showed the upper portion of all arches, extending down approximately 30 ft. from their tops, to be in perfect condition and undoubtedly as strong as at any time in their history. Likewise, a section of the bottom of each arch was little affected. In the intervening middle belt, the concrete had become a dead, inert material with little strength. All concrete in the buttresses was in perfect condition.

Holes were dug into the affected parts of the arches from their rear faces. A blow from a pick produced no ringing effect. Instead, the effect was such as would result from striking hard clay. The concrete was easily displaced, and, with little effort, holes were dug beyond the surface of the reinforcing steel to depths of approximately 18 in. (see Fig. 7), as far, in fact, as it was believed to be safe to carry such exploration, considering that at the time some water was in storage in the reservoir.

A review of conditions convinced all the engineers who made investigations that the damage was due to water from the reservoir percolating into the concrete and there freezing. Temperatures as low as − 25° Fahr. had been recorded during winter months. A review of records showed that when extremely low temperatures had occurred, the water level, in the natural operation of the reservoir, had always been drawn down 30 ft. or more from the crest of the dam, which accounted for the good condition of the concrete in the upper

* See p. 1453.

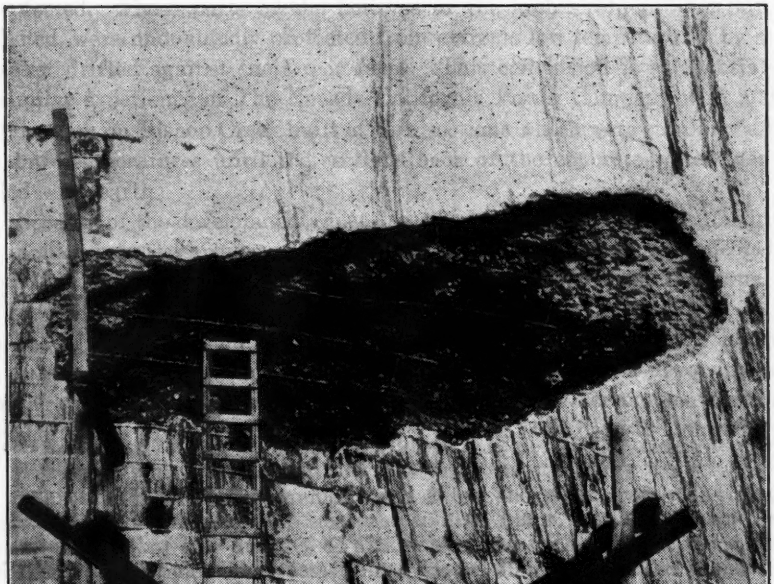

Fig. 7.—Exploration in Rear Face of Gem Dam Exposing Reinforcement.

portion of the arches. The flood stages of Rush Creek, which are due almost entirely to melting snow, occur in May, June, and July; consequently, the reservoir is drawn down before the low temperatures of the following winter occur. The buttresses, not being subjected to water pressure at any point, were unaffected. Those parts of the bottoms of the arches which were but little affected, were undoubtedly protected from extreme low temperatures by a snow blanket drifted against their rear faces. This conclusion is substantiated by a similar experience of The Nevada California Power Company with a simple intake dam on Bishop Creek built of thin sections which were similarly affected by low temperatures until the sections back of the concrete water face were filled with earth.

Because of the developments just described, it becomes of particular interest to inquire into the history and quality of the concrete which was affected. As noted by Mr. Jorgensen,* the arches were of 1 : 2 : 4 concrete, the actual ratio of sand to cement being varied with the size of the aggregates, but in all cases 1½ bbl. of cement per cu. yd. of aggregates were used. The rock was crushed in a gyratory crusher and separated into three sizes by a revolving screen having 1½, ¾, and ¼-in. meshes. The rejects from the screen went into a jaw crusher set to give a maximum size of 2 in. A sand deposit along the shore of Gem Lake was used. Mr. Jorgensen describes the sand, as follows:

"This sand was first pumped, and later shoveled, from the lake, and transported to a storage pile near the mixing plant. This lake sand, which contained 3½% of clay and 1% of dirt, was mixed with the sand from the rock crusher (all particles being less than ¼ in. in diameter) in the proportion of about three-fourths of lake sand to one-fourth of crushed rock sand. This gave a very good combination, both as to strength and water-tightness."

Compression tests on 6-in. cylinder specimens, made as the work progressed, showed an average strength of approximately 900 lb. per sq. in. for crushing at the age of 14 days. Concrete was distributed to the different arches and buttresses with two-wheeled push carts and short chutes with the minimum opportunity for separation.

Before construction was begun, the design was reviewed by the J. G. White Engineering Corporation, and a representative of this Corporation also later reviewed and favorably commented on the work in the field.

Remedial Methods

The conditions described had developed by the spring of 1924. The preceding winter had been one of almost record low precipitation. It was evident that the run-off would not be sufficient to fill the reservoir and that repairs, if carried out during this one field season, could be completed without the loss of any stored water. On the other hand, repairs extending through two seasons might necessitate wasting considerable water and thus result in a financial loss which should be directly added to the cost of repairs. It should be noted that 1 acre-ft. of water stored in this reservoir is capable of producing 1 000 kw-hr.

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 879.

Serious consideration was given to the problem of providing a rock-fill back of the concrete structure and utilizing the existing concrete arches as the water-tight face of the rock-fill dam to be thus constructed. Although a rock-fill dam, with its ready drainage, is very desirable at a location such as this, two objections to this method of repair were at once evident. The great quantity of rock necessary could not be placed in a single season, possibly not in two seasons, but, even more important, it was hardly probable that such rock-fill could be placed by any practicable methods so that it would not settle away from the concrete arches and allow them to continue to withstand the full pressure of the water until after they had actually failed.

It was finally decided that the best method of repair would be to pour a concrete gravity section back of each of the arches, extending it up to within 30 ft. of their tops. The section adopted is shown in Fig. 8. To have carried the section higher would have required greater quantities than could have been poured in a single field season. The upper 30 ft. of the existing arches and the buttresses supporting them appeared to be in perfect condition and to need no reinforcement. To clear up all doubts concerning the condition of the upper 30 ft. of the existing arches, they were drilled and tested for hardness. Two samples were cut from what appeared to be the poorest sections and were tested for strength in compression. The tests showed results of 1 880 and 2 170 lb. per sq. in., respectively. The normal operation of the reservoir during eight years had not subjected this part of the dam to water pressure during seasons of extremely low temperature, and it is not probable that future operating conditions will require that they be so exposed. It is not improbable that a horizontal crack will develop in the existing arches opposite the top of the gravity section, due to difference in rigidity of the two types, but it is not believed that any serious results will be occasioned. The two types will simply act independently and without serious leakage.

It will be noted that the section added back of the arches is not only heavier than standard profiles of gravity dams of similar height, but that, with the added pressure due to the reservoir being filled 30 ft. above the top of the gravity section, the resultant falls well within the middle third of the base, and the maximum pressure at the down-stream toe does not exceed 5 tons per sq. ft. (70 lb. per sq. in.). This is a very light pressure for the excellent hard rock foundation. In addition to the stability afforded by the gravity section, it is necessarily arched in plan, following the 40-ft. arches of the structure already existing. The section was also carried back to secure an excellent bond against the counterforts of the buttresses. It is not believed that stresses can be induced in the gravity section by pressures on the upper 30 ft. of the dam, because such pressures will be transmitted direct to the buttresses.

Drains of precast porous blocks 12 in. square with 4-in. cores were placed at the up-stream toe of the gravity section, and thus at the base of the rear faces of the arches of the former multiple-arch dam and also along the springing lines of the arches. These drains were connected to 4-in. tile-drains laid on a slope of 1 in 50.

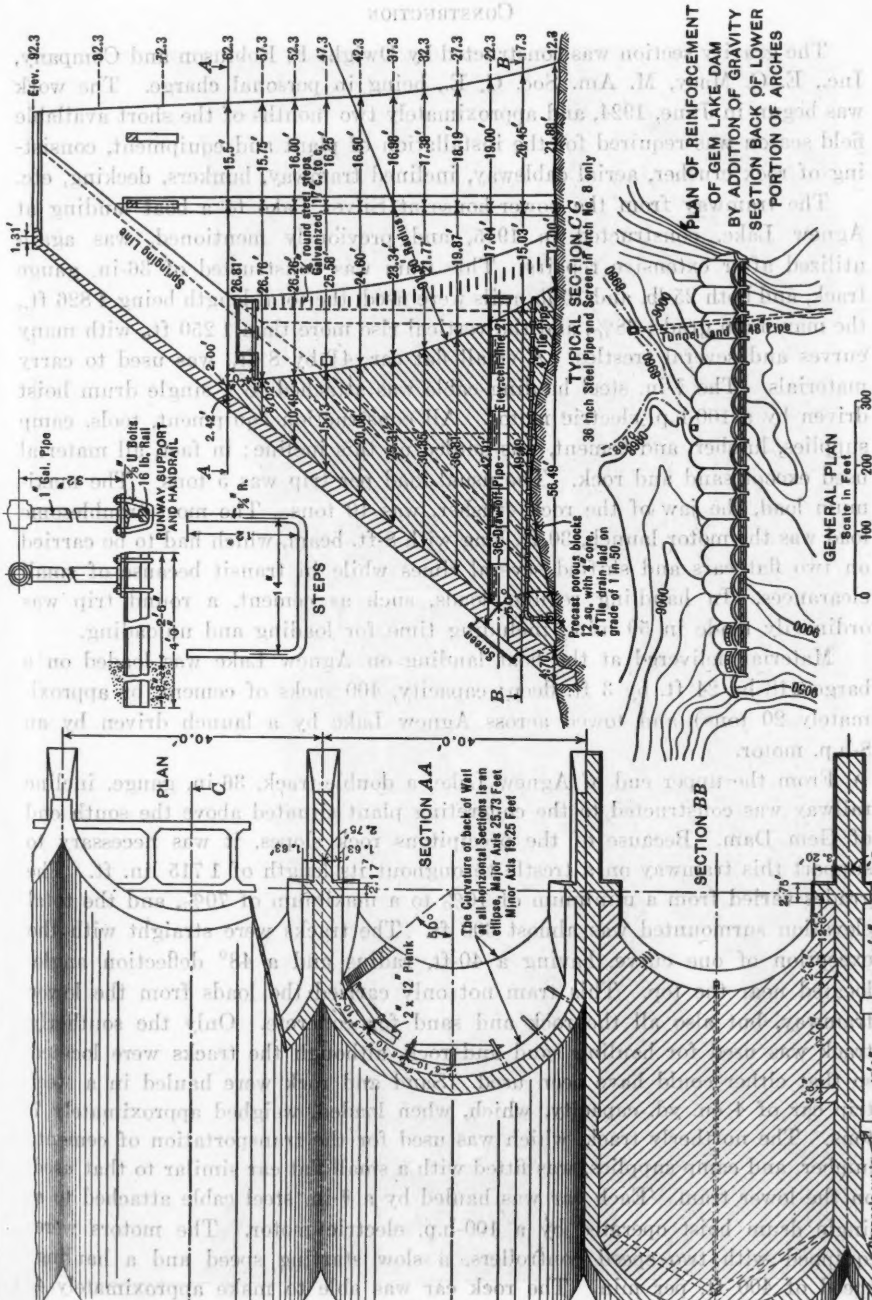
PLAN OF REINFORCEMENT
OF GEM LAKE DAM
BY ADDITION OF GRAVITY
SECTION BACK OF LOWER
PORTION OF ARCHES
TYPICAL SECTION CC
36 Inch Steel Pipe and Screen in Arch No. 8 only
GENERAL PLAN
Scale in Feet
0 100 200 300
STEPS
RUNWAY SUPPORT
AND HANDRAIL
Springing Line
Tunnel and 48" Pipe
Screen
PLAN
SECTION AA
SECTION BB
The Curvature of back of Wall at all horizontal Sections is an ellipse, Major Axis 25.73 Feet Minor Axis 19.25 Feet
Channels cut in Buttress 4" deep

FIG. 8.

Construction

The gravity section was constructed by Dwight P. Robinson and Company, Inc., E. C. Macy, M. Am. Soc. C. E., being in personal charge. The work was begun in June, 1924, and approximately two months of the short available field season was required for the installation of plant and equipment, consisting of rock crusher, aerial cableway, inclined tramway, bunkers, decking, etc.

The tramway from the power-house at Silver Lake to a boat landing at Agnew Lake, constructed in 1915, and previously mentioned, was again utilized after extensive repairs. This tram was constructed of 36-in. gauge track, and both 25-lb. and 16-lb. rails were used, the total length being 4 826 ft., the maximum grade, 68%, and the vertical rise more than 1 250 ft., with many curves and several trestles. A small flat car, $4\frac{1}{2}$ by 8 ft., was used to carry materials. The $\frac{3}{4}$-in. steel hauling cable was attached to a single drum hoist driven by a 100-h.p. electric motor. All construction equipment, tools, camp supplies, lumber, and cement, was hauled up this incline; in fact, all material used except sand and rock. The usual load per trip was 5 tons. The maximum load, the jaw of the rock crusher, was 10 tons. The most troublesome load was the motor launch, 30 ft. long with 8-ft. beam, which had to be carried on two flat cars and shifted several times while in transit because of small clearances. In handling regular loads, such as cement, a round trip was ordinarily made in 50 min., including time for loading and unloading.

Material delivered at the boat landing on Agnew Lake was loaded on a barge (18 by 24 ft. by 3 ft. deep; capacity, 400 sacks of cement, or approximately 20 tons) and towed across Agnew Lake by a launch driven by an 8-h.p. motor.

From the upper end of Agnew Lake, a double-track, 36-in. gauge, incline railway was constructed to the concreting plant situated above the south end of Gem Dam. Because of the precipitous rock slopes, it was necessary to support this tramway on a trestle throughout its length of 1 715 lin. ft. The grades varied from a minimum of 10% to a maximum of 70%, and the total elevation surmounted was almost 600 ft. The tracks were straight with the exception of one curve, having a 40-ft. radius and a 48° deflection angle, located near the top. This tram not only carried the loads from the lower tramway, but also all the rock and sand for concrete. Only the southerly track was used for hauling sand and rock, although the tracks were located so that either could have been used. Sand and rock were hauled in a steel trip car of 4 cu. yd. capacity, which, when loaded, weighed approximately 7 tons. The northerly track, which was used for the transportation of cement, lumber, and camp supplies, was fitted with a small flat car similar to that used on the lower tram. Each car was hauled by a $\frac{3}{4}$-in. steel cable attached to a single drum hoist operated by a 100-h.p. electric motor. The motors were equipped with two speed controllers, a slow starting speed and a hauling speed of 400 ft. per min. The rock car was able to make approximately 4 round trips per hour. It is estimated that 26 428 tons of materials were hauled by this double incline during the short field season.

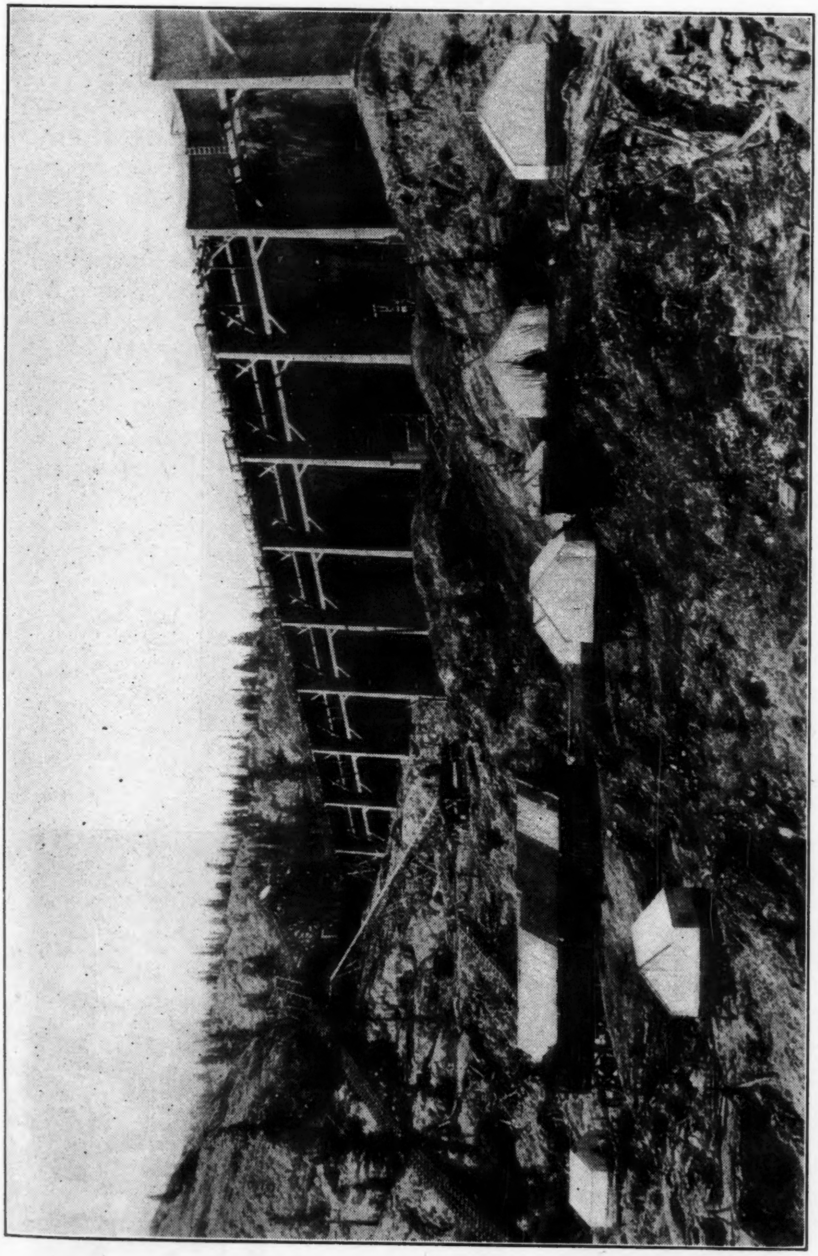

FIG. 9.—PARTIAL VIEW OF EQUIPMENT AND CAMP, SHOWING OPERATION OF INCLINED DOUBLE TRAMWAY, BUNKERS, DECKING, ETC.

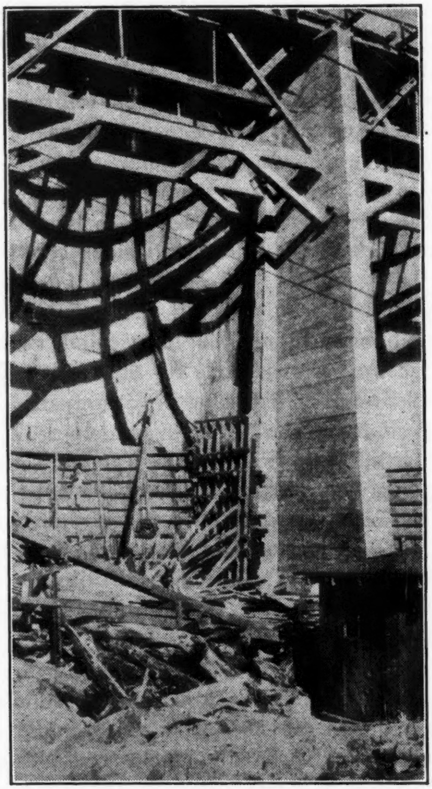

FIG. 10.—FORM WORK FOR ARCHES AND CONCRETE SPOUT FOR CONVEYING CONCRETE FROM OVERHEAD DECK.

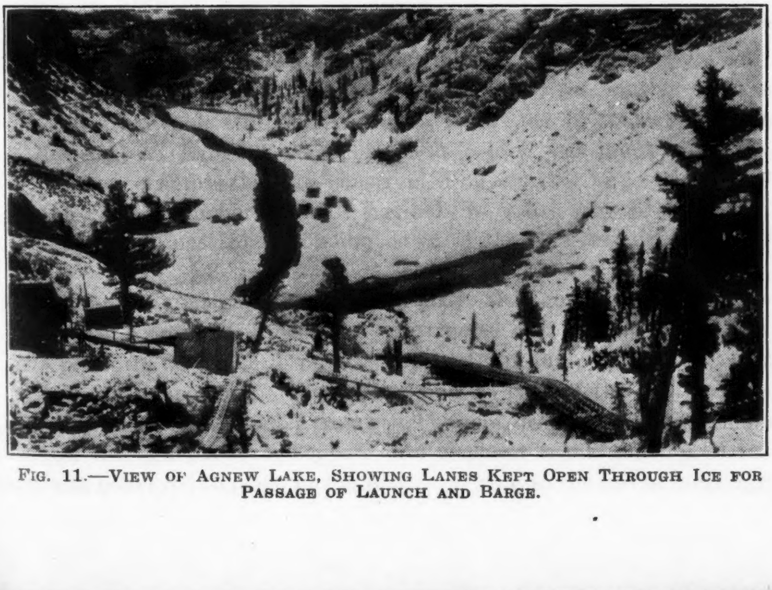

FIG. 11.—VIEW OF AGNEW LAKE, SHOWING LANES KEPT OPEN THROUGH ICE FOR PASSAGE OF LAUNCH AND BARGE.

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It was hoped that suitable rock could be found near and above the elevation of the top of Gem Dam, but final examinations showed the most suitable rock available for both the crushed rock aggregate and for making sand, to be a deposit of broken granite located below the dam and across Agnew Lake. Laboratory tests of samples of this rock showed it to be entirely satisfactory. To transport this rock across Agnew Lake, a cableway was constructed from the rock pit to the crushing plant which was at the foot of the upper incline. The rock was excavated and conveyed to the loading bunker by a $1\frac{3}{4}$ cu. yd. Bagley grader operated by a double drum hoist equipped with a 75-h.p. electric motor and $\frac{7}{8}$-in. hauling and haulback lines.

The overhead cableway between the rock pit and the rock-crushing plant required a span between supports of 950 lin. ft., the span between anchors being 1 250 lin. ft. The anchor at the end above the crushing plant comprised a set of four 2-in. eye-bolts grouted in holes drilled in a rock ledge about 300 ft. back of the plant. A single-sheave block was laced by $\frac{3}{4}$-in. wire rope to each of these eye-bolts, and a 4-sheave block was laced to another single-sheave block acting as a thimble for attaching the carrying cable. The 4-sheave block was about 50 ft. from the eye-bolts and was laced continuously through each of these single-sheave blocks with the loose end of the cable clipped back on itself. Whenever an adjustment for height was necessary, this loose end was attached to the hauling line for the cableway car and any slack was taken up by power.

The anchor back of the rock pit consisted of a "deadman" and head-frame of round timbers backfilled over with about 100 tons of rock. The dead end of the cable was clipped around a 36-in. log and carried up through the head-frame, where a 6-sheave block allowed adjustments for height. The hauling hoist was on the mountain side to the rear of the rock-crushing plant and directly under the carrying cable. It was designed to operate the car at a line speed of 500 ft. per min. The haulback hoist was set in the head-frame of the rock-pit anchor. The erection of the original carrying cable, which was $2\frac{1}{4}$ in. in diameter, was a difficult task requiring the laying of the cable across the lake by utilizing the barge and motor, attaching the end at the rock-pit anchor, and pulling it up to position by use of the hauling hoist and rope. The original cable stranded soon after it was put in operation and was replaced by a $1\frac{1}{2}$-in. Langlay, steel-core cable, which was much more easily handled. Pontoons across the lake, constructed by securing in position beneath the carrying cable a number of small logs held in position by $\frac{3}{8}$-in. wires, prevented the hauling lines from dropping down in the lake and catching under rocks.

The concrete mixing plant was located above the south end of Gem Dam. From this point a trestle and deck was constructed over the top of the dam to serve as a platform on which to push the concrete in dump cars of the Kopple type to the point of pouring. Fig. 9 is a view of part of the equipment and camp, showing the inclined double tramway, bunkers, decking, etc. The concrete was dumped through platform hoppers into elephant trunk chuting 16 in. in diameter and conveyed to place in the forms. Owing to the use of large aggre-

gate, it was difficult to handle concrete which showed a slump of less than $2\frac{1}{2}$ in. without clogging the chutes, even with the steep slopes used. A concrete inspector made frequent slump tests and attempted to keep all the concrete of such consistency as to show a little less than a 3-in. slump. A cement bag shaker was utilized and 1 sack of cement was recovered from each 58.4 sacks shaken.

A total quantity of 12 004 cu. yd. of concrete was added to the dam and 16 425 bbl. of cement, or 1.368 bbl. per cu. yd. of concrete, were used. The approximate proportions of the mix were 1 part cement, 3 parts sand, 3 parts crushed stone ($\frac{1}{4}$ in. to $1\frac{1}{2}$ in.) and 1.84 parts cobbles. Compressive tests of concrete samples showed an average strength of 2 400 lb. per sq. in. at an age of 28 days, and 3 440 lb. per sq. in. at an age of 3 months.

Weather conditions permitted continuance of the work somewhat longer than usual, and the last concrete was poured on November 15, 1924. Fig. 10 shows the form work for the arches and the concrete spout for conveying concrete from the overhead deck. Fig. 11 is a view of Agnew Lake showing the lane kept open through the ice for the passage of the launch and barge.

Conclusions

To prevent deterioration of concrete subjected to water pressure and extreme low temperatures, it is necessary that impervious concrete be obtained. The slightest penetration of water will be followed by deterioration, further penetration, and further deterioration.

Plaster coats, such as "Ironite", might be satisfactory except for hair-line cracks which will be developed on any thin arch section by movement due both to temperature changes and to loading and unloading the arches.

Under the difficulties imposed at this site and similar sites in very remote high mountain areas, it is doubtful whether, under conditions of the present day, impervious concrete can be obtained. Therefore, structures composed of thin sections of concrete should be protected from extremely low temperatures. For such localities, the writers believe that a dam of the rock-fill type has many advantages over dams of other types.

Acknowledgment

In addition to the writers, those who reviewed conditions and assisted in planning the reconstruction include Messrs. R. G. Manifold and the late Charles Oscar Poole, M. Am. Soc. C. E., Consulting Engineers, in whose office the plans for reconstruction were made, and C. W. Comstock, M. Am. Soc. C. E. The construction work was under the general charge of E. J. Waugh, M. Am. Soc. C. E., Construction Engineer of The Nevada California Power Company. As previously noted, this work was performed by Dwight P. Robinson and Company, Inc., with E. C. Macy, M. Am. Soc. C. E., in personal charge.

POWER DEVELOPMENT IN THE MIDDLE WEST*

By W. L. Abbott,† Esq.

It is a mooted question, where the West does begin. Perhaps along the crest of the Continental Divide, perhaps on its western slope, but more likely where the rich verdure of the prairies fades into the dull gray of the plains; that is where the West begins. Eastward from a line just beyond the best parts of the Dakotas, Nebraska, and Kansas, lies a fertile valley, 1 000 miles in width, known to the West as "back East", sometimes as "back home", known to the East as "out West", sometimes as the "Middle West".

The territory comprised in this survey includes and extends from Western Pennsylvania to Eastern Kansas, and from Southern Kentucky to the Great Lakes and the Canadian border. It comprises an area of 660 000 sq. miles and a population of 40 000 000. It all lies in the Mississippi Valley and scarcely 1% can be rated as mountainous, its broad stretches of prairie and forest lands having in general such slight declivity and absorbent soil that the run-off, except that lost in floods, does not exceed one-fifth the precipitation, and its placid streams for the greater part of their courses flow to the Lakes or sea with a slope of ½ ft. to the mile.

Such a gentle fall makes possible relatively little water power, considering the length and breadth of the streams, but as compensation, because these streams have not eroded the soil, the prairie States, with their unrivaled fertility, will some day support a population of hundreds of millions whose increasing clamor for power must be satisfied principally, as now, by coal drawn from their own great stores.

Fig. 1 shows the various States in their true importance electrically. This map, which shows relative kilowatt-hour production, is a fairly accurate representation of installed kilowatt capacity, the total for the United States being given as 14 000 000, of which about 4 000 000 kw. is hydro-electric.

In this classification, as in many others, New York stands at the head, with 40% of its output from its water powers, principally those at Niagara. Next, comes Pennsylvania; then California, 80% of the electrical output of which is generated by water power from its impounded mountain streams. Illinois is fourth, 95% of its output coming from coal. Ohio is fifth, with 99% of its output also from coal. Michigan is sixth with 32% of its output from water. West Virginia, a mountainous State, is ninth, with only 2% of its output from water power. Kentucky, also a mountainous, coal State,

* Presented at the meeting of the Power Division, Chicago, Ill., July 11, 1923.
† Chf. Operating Engr., Commonwealth Edison Co., Chicago, Ill.

has developed practically no water power. Idaho and Montana, in the middle of the list, obtain less than 1% of their power from fuel.

The various potential power streams of the United States and the water powers are generally situated where they will do the most good, that is, remote from coal mines, the Middle West, in comparison with the remainder of the country, not being particularly favored. The States of Ohio, Indiana, and Illinois, which are richest in coal, are poorest in water power. Only 18%, or 776 000 kw., of the water power of the country lies in the Middle Western States, and the greater part of this is in Michigan, Wisconsin, and Minnesota.

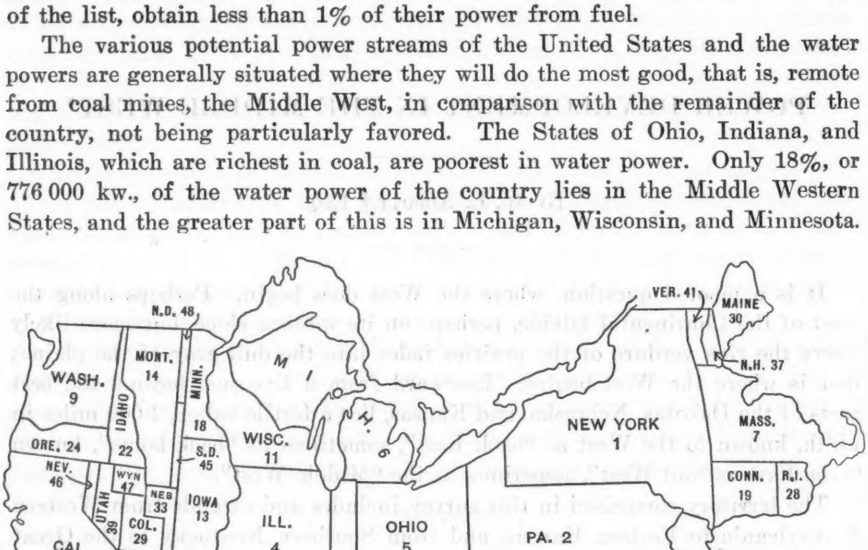

FIG. 1.—OUTLINE MAP OF UNITED STATES, SHOWING RELATIVE KILOWATT-HOUR PRODUCTION OF THE VARIOUS STATES.

Water power must be generated at the water-falls, and until long-distance transmission was developed that power had to be used where it was generated; and because much of such power was in rough, inaccessible country, having no other features of commercial importance, such rivers were allowed to roll on in solitude "and hear no sound save their own dashing".

Electric transmission has changed all this, and whereas once factories went to the power, now power goes to the factories, and with a widening effective range of transmission lines and higher cost of coal, more and more of these wild waters are being broken to harness and few of those remaining are escaping serious considerations.

Successful hydro-electric long-distance power transmission naturally suggests that steam-electric power be generated at the mine and sent by wire, instead of by rail, to the remote centers where it is to be used, thus circumventing the railroads and their freight bills. For some reason it is not done in any large way, the obvious explanation being that, with summer temperatures, 500 lb. of circulation water are required per pound of coal burned, and with a plant of 50 000 kw. about 200 cu. ft. of water per sec. would be required for condensation, and streams of that flow in summer rarely

abound on the prairies near coal mines. This is the obvious and the usual reason why steam-electric power is not developed in quantity at the mines and transmitted long distances by wire. Although this answer is usually correct and sufficient, it is not always so, as there are many places where coal is produced on the banks of large rivers and yet the coal leaves the mine by rail for power plants 100, 200, or 300 miles away. It must be concluded, therefore, that rail transmission of power is still cheaper than wire transmission and, curiously enough, the longer the distance the less advantageous becomes electric transmission.

To illustrate, compare a water power capable of generating 100 000 kw., with a coal mine, each 50 miles from a large market for power. Assume, also, that a steam-electric plant can be built for $120 per kw. of capacity and that the water power can be developed and the energy transmitted over a double-tower line of two circuits, the right of way for each line costing $35 000 per mile. Assume that the coal can be bought at the mine for $2.50 per ton, and that 1 kw-hr. can be generated on 1.8 lb., assuming an annual load factor of 75 per cent.

What are these relations at 100 miles and at 200 miles? The problem is: Shall the water power be developed and the energy transmitted, or shall the steam plant be built at the coal mine and the energy transmitted, or shall a steam plant be constructed where the power is to be used and the coal shipped to it from the mine?

It may also be of interest to railroad men to have a layman explain the system by which railroad coal freight schedules are made up. The writer does not know that there really is a system by which these schedules are constructed, but the following rule will loosely fit most of them in this locality: A charge of 0.4 cent per ton-mile, plus a base rate of $1.00 per ton. For example, from Braidwood, Ill., to Chicago is 57 miles; multiplying by 0.4 cent gives 23 cents and adding the base rate of $1.00 equals $1.23. The actual rate is $1.48. From Danville, Ill., to Chicago is 123 miles. By the formula the rate whereas it actually should be $1.49 is $1.55. Springfield, Ill., 185 miles away, should have a rate of $1.74; it is $1.65. Harrisburg, Ill., 306 miles distant, should have a rate of $2.24; it is $2.16. Applying this freight rate formula to the problem previously stated, the results given in Table 1 are obtained.

From Table 1 it appears that with coal at prevailing prices in this territory, water power within 200 miles of the place where its power can be used, has some intrinsic value as such, but beyond that distance it had better be developed for its scenic value. For a power plant at the mine and with an abundance of condensing water at hand, it too would be outclassed by a power-house supplied by rail-borne coal, situated in the big city 100 miles or more from where the power is to be used. If the hydro-electric plant or the steam plant at the mine were obliged to carry enough of the load to reduce its load factor to 50%, it is probable that all three would be on about the same basis at 50 miles, and beyond that distance the plants operating over transmission lines would begin to have an advantage over the plant situated near the place where its power is to be used.

TABLE 1.

	Hydro-electric plant.	Steam plant, mine.	Steam plant, city.
50-Mile Line:			
Cost of power plant........................	$20 800 000	$12 600 000	$12 000 000
Cost of line........................	1 750 000	1 750 000	....
Total........................	$22 550 000	$14 350 000	$12 000 000
Overhead at 15%........................	$3 380 000	$2 150 000	$1 800 000
Coal consumed, in tons per year........................	....	603 000	592 000
Mine cost........................	....	1 507 000	1 480 000
Freight........................	....	....	710 000
Cost of fuel delivered per year........................	....	1 507 000	2 190 000
Fuel and overhead........................	$3 380 000	$3 657 000	$3 990 000
100-Mile Line:			
Cost of power plant........................	$21 500 000	$13 100 000	$12 000 000
Cost of line........................	3 500 000	3 500 000	....
Total........................	$25 000 000	$16 600 000	$12 000 000
Overhead at 15%........................	$3 750 000	$2 490 000	$1 800 000
Coal consumed, in tons per year........................	....	622 000	592 000
Mine cost........................	....	1 555 000	1 480 000
Freight........................	....	....	829 000
Cost of fuel delivered per year........................	....	1 555 000	2 209 000
Fuel and overhead........................	$3 750 000	$4 045 000	$4 009 000
200-Mile Line:			
Cost of power plant........................	$23 000 000	$14 200 000	$12 000 000
Cost of line........................	7 000 000	7 000 000	....
Total........................	$30 000 000	$21 200 000	$12 000 000
Overhead at 15%........................	$4 500 000	$3 180 000	$1 800 000
Coal consumed, in tons per year........................	....	651 000	592 000
Mine cost........................	....	1 627 000	1 480 000
Freight........................	....	....	1 066 000
Cost of fuel delivered per year........................	....	1 627 000	2 546 000
Fuel and overhead........................	$4 500 000	$4 807 000	$4 346 000

Fig. 2 shows the distribution lines of the territory discussed. The lines and circles show the manufacturing centers and the districts of greatest population, beginning at Pittsburgh, Pa., on the east, following along the Lakes to Detroit, Mich., and then across the Lower Michigan Peninsula, through a number of flourishing manufacturing towns, to the south end of Lake Michigan, where the various power plants of the Commonwealth Edison Company are shown in the area of one large circle, which is the dominating feature of the whole map. Within the circle, but not included in its area, are circles denoting various other power-houses in the Chicago District.

Farther north is a circle which includes the combined power of Milwaukee and Racine, Wis., together with some energy received over a high-tension line

from the water-power development on the Wisconsin River at Kilbourn and Prairie du Sac, Wis. Farther northwest is a larger circle, denoting the power of Minneapolis, Minn., and a smaller one within it, showing the generating capacity in St. Paul, Minn., and north and east is the combined power of Duluth, Minn., and West Superior, Wis., with some energy fed in over a transmission line from water powers farther northwest.

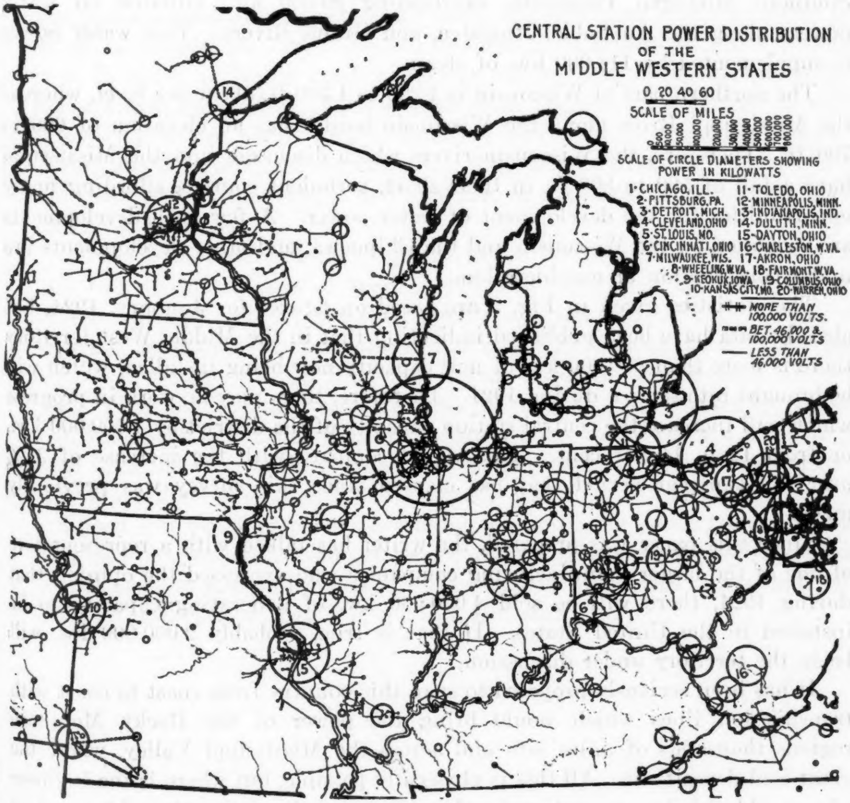

FIG. 2.

Going down the Mississippi River, the first development of any consequence is that of the tri-cities—Davenport, Iowa, and Moline and Rock Island, Ill.—and farther down stream is the Keokuk, Iowa, water power, shown by a broken circle. The greater part of this power is sent to St. Louis, Mo., over a 110 000-volt transmission line, and is included in the area surrounding that city. The circle showing the installed generating capacity at Kansas City, Mo., which surrounds the circle indicating the power installed at Kansas City, Kans., Joplin, Mo., and Wichita, Kans., is a feature in the southwest corner of the map (Fig. 2). In Indiana, Indianapolis, New Albany, and Fort Wayne are the cities using the most power. In Ohio, power is more evenly distributed among its numerous thriving manufacturing cities than in any other State. All over Ohio, Indiana, Michigan, and Iowa, and, to a lesser extent, Wis-

consin and Minnesota, the small towns and villages are connected by tie lines of less than 40 000 volts, fed from some centrally located power system in one of the larger towns.

Of this whole territory, about one-sixth of the power is hydraulic and five-sixths steam, the largest single water power being that at Keokuk, which is more than 100 000 kw. The next largest series of water powers is in the Southern Michigan Peninsula, aggregating 86 000 kw., situated on water powers along the Au Sable, Manistee, and Grand Rivers. This water power is supplemented by 114 000 kw. of steam.

The northern part of Wisconsin is 1 200 to 1 500 ft. above sea level, whereas the Mississippi River along the Wisconsin border has an elevation of 600 to 700 ft. Therefore, the Wisconsin rivers which discharge into the Mississippi have a fall of 500 to 800 ft. in their short, turbulent courses, affording many opportunities for the development of water power. A few such developments are indicated on the Wisconsin and the Chippewa, and other developments are under construction or consideration.

The statistics given in Fig. 2 are based on figures for January, 1924, but already data have been published indicating that in the Middle West territory there is more than 800 000 kw. of new capacity now being installed, which will be brought into service during 1924. Therefore, there may be work in progress which will increase the central station capacity of the district by 1 000 000 kw., or from 12 to 14 per cent. This would compare with the increase of 31% over the two-year period that has elapsed since this survey was previously made.

Since this paper was prepared, the writer has talked with a representative of one of the larger manufacturing companies who expressed the opinion that during 1924, there will be sold 4 000 000 kw. of generating capacity to be installed in the United States. If that is true, probably 2 000 000 kw. will be in the territory under discussion.

It has been seriously proposed to span this country from coast to coast with transmission lines which would bring the power of the Rocky Mountain regions thousands of miles into and across the Mississippi Valley, where the great coal deposits lie. All this is physically possible, but where is the engineer who would risk his reputation by thus recommending that the white coal of the mountains be carried to the black coal fields of the Middle West?

There is great promise in the newly developed magnetron tube, a simple device by which alternating current may be economically converted to direct current of equivalent voltage, or *vice versa.* This may make it possible in transmission to increase line voltages greatly and reduce the size of the conductor, and, all things considered, render it feasible to transmit, with high-voltage direct current, double the power on the same size of conductor that is now possible with alternating current; so that on a double-tower line of six conductors each, as much as 1 000 000 kw. may be transmitted, at a loss of between 5 and 10% for each 100 miles. Even with that advantage, however, the Middle West must continue to draw its own energy from its well-nigh inexhaustible coal beds and develop its power systems, as in the past, by great

power-houses near the centers where power is needed, linked together by tie lines which, although not capable of carrying any considerable amount of the power needed in any large center, will serve nevertheless as a means for cutting off peaks, equalizing load factors, and carrying some parts of the load in times of distress.

At present, the central station industry is prosperous and is growing as fast as money can be found for financing. All over the country under discussion new power-houses are projected and under construction, with capacities from 40 000 to 100 000 kw., and designed to serve the territory within a radius of 50 to 75 miles, replacing smaller and less efficient generating apparatus in all towns within that radius. In the larger cities there is great activity in adding to existing power plants and in building new ones of capacities which far exceed anything hitherto constructed. The Middle West is being electrified and more and more strongly tied together, to take care of an ever-increasing volume of business.

It has been predicted that central station business will soon divide itself into two distinct classes of service: (1) The generation and wholesaling of electric energy; and (2) the distribution and retailing of this energy, with separate companies for each class of business. Be that as it may, these growing power systems will eventually be casting about for greater fields of usefulness. They may then find that the same railroads that are crossed by transmission lines and that bring fuel to the power-houses may be in a mood to part with their old-time faithful friend, the puffing locomotive, and modernize. If and when that time comes, the railroads will find at hand, organized and equipped, a network of nerves, extending all across the countryside, into which they can plug anywhere and get power, and which can readily be expanded, as in the past, to meet all demands. Destiny has willed this union, and on all sides are indications that it is coming, and soon.

THE IMPROVED VENTURI FLUME

By Ralph L. Parshall,* Affiliate, Am. Soc. C. E.

Synopsis

The device herein described is a promising structure for measuring the rate of flow of either small or large quantities of water. It has no moving parts, and is simple and inexpensive. It utilizes the principle of a hydraulic control and sacrifices little head. Tests indicate an accuracy of about 5 per cent.

Requirements to Be Met.—The measurement of flowing water in irrigation practice often requires that little loss of head be sacrificed by the measuring device. Because of the relatively large loss of head, the standard overfall weir is, in many cases, objectionable. Although when operating under standard conditions, it is one of the most accurate of measuring devices for practical field installation, it is not always possible to maintain these standard conditions, such as proper bottom contraction, velocity of approach, and aeration of nappe. The submerged orifice has not met with entire success because of the uncertainty of the coefficient of discharge, velocity of approach, and constancy of condition. The ordinary rating flume often becomes inaccurate within a short time.

To meet the objections found in other measuring devices now in common use, the present Improved Venturi Flume has been developed. The improvements over the old Venturi flume consist in the reduction of the convergence in the inlet section, lengthening of the throat section, change of divergence of the outlet section, and depressing the floor in the throat section.† These changes have improved the flow conditions, reduced the effect of submergence, and simplified the operation by reducing the number of gauges necessary to determine the discharge.

Type of Structure.—This flume (Fig. 1) has three major parts, a converging inlet, a parallel-sided throat, and a diverging outlet. All the side-walls are vertical. The inlet has a level floor between the sides, each side converging at a rate of 1 ft. in 5 ft. of length. The floor of the throat slopes

Note.—The Special Committee on Irrigation Hydraulics selected the subject of "Measuring Irrigation Deliveries" as one of ten for study and research. This paper was submitted to the Committee by its author, and the Committee has recommended its publication in *Proceedings*, in order to elicit discussion of this subject. (See Progress Report of the Committee, *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, p. 137.)

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† "The Venturi Flume", by V. M. Cone, *Journal of Agricultural Research*, Vol. IX, No. 4, 1917; "The Venturi Flume," by R. L. Parshall and Carl Rohwer, *Bulletin 265*, Colorado Agricultural Experiment Station, 1921.

downward with a vertical fall of 9 in. in a horizontal distance of 2 ft. The outlet floor slopes upward at the rate of 6 in. in 3 ft., and each wall diverges at the rate of 1 ft. in 6 ft. of length. The crest is the down-stream end of the level floor, and the crest length or size of flume is the distance, in feet, between the vertical walls of the throat. The lengths of the throat and the outlet of the flume are 2 and 3 ft., respectively, for all sized flumes from 1 ft. to 8 ft. The side of the inlet is made longer as the width of the flume increases, according to the arbitrary rule, $\frac{W}{2} \div 4$, in which, W is the width of the flume, or crest length, in feet.

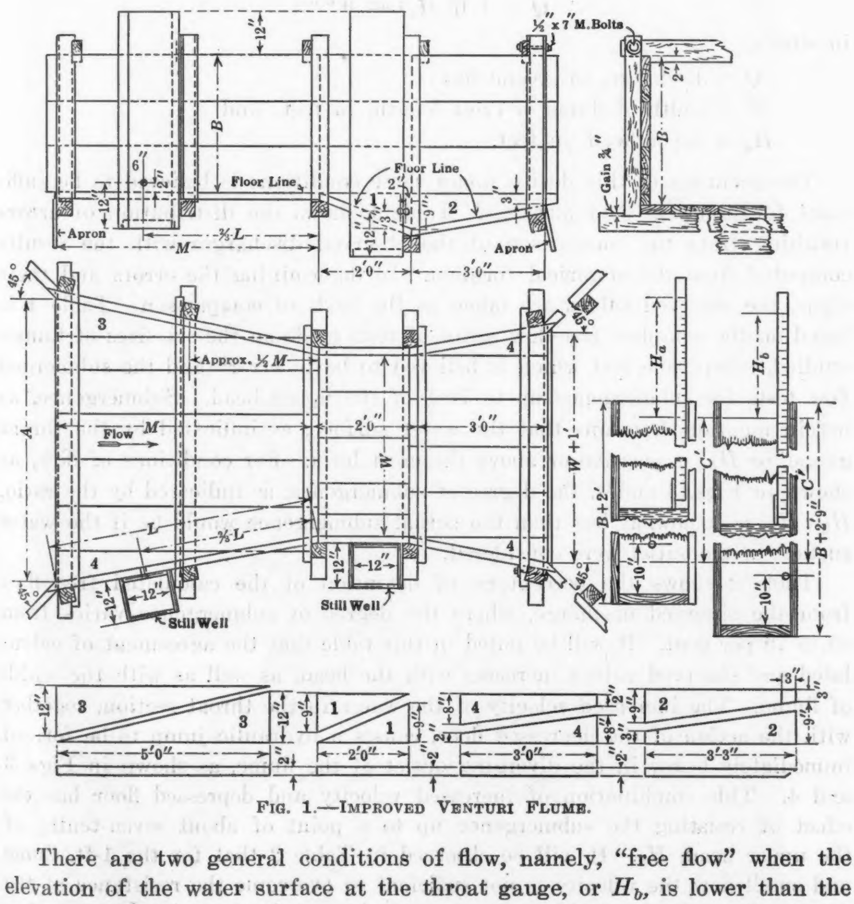

FIG. 1.—IMPROVED VENTURI FLUME.

There are two general conditions of flow, namely, "free flow" when the elevation of the water surface at the throat gauge, or H_b, is lower than the crest, and "submerged flow", when the elevation of the water at the throat gauge is higher than the crest. The upper head, H_a, is observed at a point two-thirds the distance from the crest to the upper end of the inlet, along the side-wall, whereas H_b, the throat head, is observed at a point 3 in. vertically and 2 in. horizontally up stream from the lowest point of the depressed throat

floor. Both the upper head, H_a, and the throat head, H_b, are referred to the crest level as the datum. Under certain conditions of flow the throat reading may be negative, as shown in Fig. 2.

Laboratory Tests.—Calibrations were made on flumes having crest lengths of 1, 2, 3, 4, 6, and 8 ft., with a minimum flow of 0.3 sec-ft. through the 1-ft. flume, and a maximum of 62.5 sec-ft. through the 6-ft. flume, the supply not being available to exceed this maximum through the 8-ft. flume. The discharge in second-feet through this device, under free-flow conditions and under submerged conditions when the down-stream head does not exceed 70% of the upper head, is very closely approximated by the empirical formula:

$$Q = 4\ W\ H_a^{1.522\ W^{0.026}}$$

in which,

Q = discharge, in second-feet;
W = width of flume or crest length, in feet; and
H_a = upper head, in feet.

The accuracy of this device under these conditions is believed to be sufficient for practical field purposes. Table 1 shows the distribution of errors resulting from the comparison of the observed discharges with the results computed from the empirical formula. In determining the errors and their signs, the observed values are taken as the basis of comparison. Table 1 is based on the complete free-flow series of tests made on the six sizes of flumes studied, except one test which is believed to be in error* and the submerged flow tests for submergence up to 71% of the upper head. Submergence, as herein mentioned, means that the water surface, as indicated by the throat gauge, or H_b, is positive or above the crest level. For conditions of flow, as shown in Figs. 3 and 4, the degree of submergence, as indicated by the ratio, $H_b : H_a$, is somewhat less than the actual submergence would be if the water surface in the outlet were considered.

Table 2 shows the percentage of deviation of the calculated free flow from the observed discharge, where the degree of submergence varies from 69 to 79 per cent. It will be noted in this table that the agreement of calculated and observed values increases with the head, as well as with the width of flume. The increased velocity of the water in the throat section, together with the action of the depressed floor, causes a hydraulic jump to be formed immediately below in the diverging outlet of the flume, as shown in Figs. 3 and 4. This combination of increased velocity and depressed floor has the effect of resisting the submergence up to a point of about seven-tenths of the upper head, H_a. It will be observed in Table 2 that for the 1-ft. flume and small flow the velocity is not sufficient to overcome the resistance of the water held in the depressed section, or, in other words, the point of measuring the throat head, H_b, is too far down stream to be within the range of the draw-down of the water surface for these small heads.

*Test No. 6512 on the 3-ft. flume, in which, H_a = 0.705 ft.; H_b = 0.264 ft.; percentage of submergence = 0; observed quantity = 8.33 sec-ft.; computed quantity = 6.94 sec-ft.; percentage of error = 16.7.

FIG. 2.—FREE-FLOW DISCHARGE, 6-FOOT IMPROVED VENTURI FLUME.

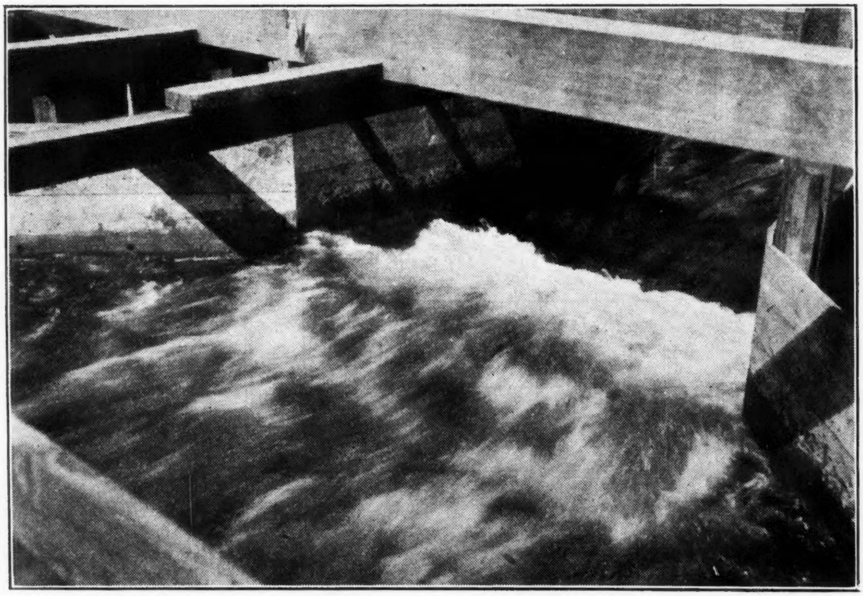

FIG. 3.—FREE-FLOW DISCHARGE, 6-FOOT IMPROVED VENTURI FLUME, WITH APPROXIMATELY 75 PER CENT. SUBMERGENCE.

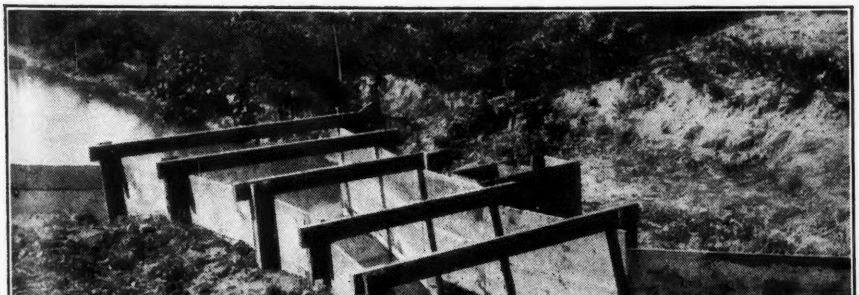

FIG. 4.—EXPERIMENTAL 3-FOOT IMPROVED VENTURI FLUME AT WELLINGTON, COLO.

TABLE 1.—SHOWING NUMBER OF OBSERVATIONS AND DISTRIBUTION OF ERRORS, FREE-FLOW AND SUBMERGED FLOW UP TO 71 PER CENT. OF UPPER HEAD.

IMPROVED VENTURI FLUME.

Size of flume, in feet.	Total obser-vations.	Percentage of observations under 5% error.	Percentage of Error. 0–0.9	1.0–1.9	2.0–2.9	3.0–3.9	4.0–4.9	5.0–5.9	6.0–6.9	7.0–7.9	8.0–8.9	9.0–9.9	10.0–10.9	11.0–11.9
			Number of Observations.*											
1	32	75	+6 −9	+5 −1	+3 −0	+1 −0	+3 −0	+2 −0	+0 −0	+1 −0	+0 −0	+0 −0	+0 −0	+1 −0
2	31	81	+6 −4	+5 −8	+2 −0	+4 −0	+1 −0	+0 −0	+0 −0	+1 −0	....	....	....	....
3	33	94	+5 −12	+4 −6	+2 −2	+1 −0	+1 −0	....	....	....	....	....	....	....
4	19	95	+3 −6	+4 −4	+0 −1	+0 −1	....	....	....	....	....	....	....	....
6	14	100	+2 −1	+1 −6	+0 −4	....	....	....	....	....	....	....	....	....
8	23	96	+5 −12	+1 −3	+1 −0	+0 −0	+0 −0	+0 −0	+0 −0	+0 −0	+1 −0	....	....	....

* The figures preceded by plus or minus signs show the number of tests, within the limits indicated, in which the computed values are, respectively, greater or less than the observed values.

TABLE 2.—Comparison of Calculated Free-Flow with Observed Discharge for Submergence Varying from 69 to 79 Per Cent.

Test No.	H_a, in feet.	H_b, in feet.	Percentage of submergence.	Observed discharge, in second-feet.	Calculated discharge, in second-feet.	Difference.	Percentage of deviation.
				1-Foot Flume.			
6707	0.200	0.141	70.5	0.31	0.35	0.04	12.9
6703	0.398	0.276	69.3	0.91	0.98	0.07	7.7
6667	0.600	0.424	70.7	1.74	1.84	0.10	5.7
6699	0.599	0.448	74.8	1.71	1.84	0.13	7.6
6695	0.801	0.568	70.9	2.72	2.85	0.13	4.8
6694	0.802	0.605	75.4	2.69	2.86	0.17	6.3
6710	1.002	0.708	70.7	3.86	4.01	0.15	3.9
6689	1.001	0.759	75.8	3.78	4.01	0.23	6.1
6686	1.202	0.857	71.3	5.10	5.29	0.19	3.7
6675	1.603	1.200	74.9	7.73	8.20	0.47	6.1
				2-Foot Flume.			
6645	0.201	0.140	69.7	0.62	0.66	0.04	6.5
6646	0.201	0.154	76.6	0.61	0.66	0.05	8.2
6637	0.402	0.281	69.9	1.87	1.95	0.08	4.3
6638	0.402	0.298	74.1	1.82	1.95	0.13	7.1
6631	0.598	0.421	70.4	3.49	3.60	0.11	3.2
6632	0.602	0.456	75.7	3.38	3.64	0.26	7.7
6606	1.002	0.707	70.6	7.89	8.02	0.13	1.7
6607	1.000	0.768	76.8	7.70	8.00	0.30	3.9
6612	1.602	1.127	70.3	16.51	16.61	0.10	0.6
6613	1.593	1.217	76.4	15.89	16.47	0.58	3.7
				3-Foot Flume.			
6579	0.199	0.140	70.4	0.92	0.96	0.04	4.3
6575	0.298	0.221	74.2	1.71	1.80	0.09	5.3
6528	0.399	0.298	74.7	2.75	2.85	0.10	3.6
6520	0.503	0.379	75.4	3.93	4.09	0.16	4.1
6534	0.595	0.446	75.0	5.13	5.32	0.19	3.7
6556	0.600	0.466	77.7	5.13	5.39	0.26	5.1
6514	0.699	0.499	71.4	6.74	6.85	0.11	1.6
6553	0.900	0.647	71.9	10.09	10.17	0.08	0.8
6543	0.996	0.710	71.3	11.84	11.93	0.09	0.8
6561	0.999	0.769	77.0	11.92	11.98	0.06	0.5
6563	1.000	0.754	75.4	11.81	12.00	0.19	1.6
6544	1.004	0.799	79.6	11.79	12.08	0.29	2.5
6572	1.200	0.836	69.7	15.99	15.96	—0.03	0.2
6411	2.039	1.509	74.0	35.23	36.62	1.39	3.9
6408	2.358	1.697	72.0	45.66	45.98	0.32	0.7
				4-Foot Flume.			
6399	1.014	0.791	78.0	15.09	16.35	1.26	8.4
6395	1.339	0.977	73.0	24.55	25.36	0.81	3.3
6373	1.702	1.293	76.0	35.44	37.03	1.59	4.5
6382	2.008	1.475	73.5	46.31	48.07	1.76	3.8
				6-Foot Flume.			
6359	0.900	0.678	75.3	20.11	20.29	0.18	0.9
6353	1.143	0.808	70.8	30.04	29.71	—0.33	1.1
6354	1.171	0.905	77.3	29.94	30.87	0.93	3.1
6347	1.396	1.065	76.3	40.20	40.86	0.66	1.6
6339	1.562	1.120	71.7	49.51	48.88	—0.63	1.3
6340	1.617	1.273	78.7	50.07	51.66	1.59	3.2
				8-Foot Flume.			
6313	1.064	0.774	72.7	35.08	35.36	0.28	0.8
6304	1.246	0.871	69.9	45.12	45.57	0.45	1.0
6305	1.270	0.937	73.8	45.23	46.97	1.74	3.8

For the most satisfactory operation of this device, it is recommended that the flume operate under conditions in which the submergence does not exceed 70 per cent. Under such conditions only one observation of depth is necessary to determine the rate of flow. For submergence greater than 70 to 75%, the discharge is then a function of both the upper head, H_a, and the throat head, H_b. Laboratory tests show that when the submergence is greater than about 98%, no great dependence can be placed on the measurement.

Field Tests.—Since completing the calibrations of the various sized flumes, outside field installations have been made to test the device under practical conditions. At Wellington, Colo., a small lateral was selected having a flat grade, where the stream of water carried some sand and silt. In Fig. 4 is shown an experimental 3-ft. Improved Venturi Flume in this lateral, where the upper head, H_a, is 0.86 ft., and the throat head, H_b, 0.63 ft., with a difference of 0.23 ft., the ratio, $H_b : H_a$, being 73 per cent. The discharge under this condition is in accordance with the free-flow law and the rate of flow can be assumed to be a function only of the upper head, H_a, thus giving a discharge of 9.48 sec-ft. A standard weir will not operate successfully in this lateral, first, because of the deposit which would occur in the weir box; and, second, because of the small head available. This discharge of 9.48 sec-ft. over a 3-ft. rectangular weir would require a head of exactly 1 ft., and to this possibly an extra 0.10 ft. should be added to assure a complete aeration, making a total loss of head of about 1.1 ft. By careful measurements with an engineer's level, the loss of head through the Improved Venturi Flume, under the conditions as stated, was only 0.19 ft.*

Under conditions where sand and silt occur, it has been observed that if a difference in head of 0.05 to 0.10 ft. can be maintained for a moderate upper head, no deposit will accumulate in the flume even where excessive quantities of sediment are carried in the stream.

Velocity of approach seems to have little or no effect on the rate of discharge for free-flow conditions.

Pertinent Features.—From the standpoint of measuring irrigation water, the Improved Venturi Flume has the following desirable features.

1.—It is sufficiently accurate for practical purposes.

2.—It operates in sand or silt-laden streams without trouble.

3.—It operates successfully with relatively small loss of head.

4.—It is able to withstand a high degree of submergence.

5.—It maintains constancy of condition.

6.—It possesses simplicity of operation.

7.—There are no moving parts.

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* Note approximately similar conditions of heads for the 3-ft. flume, Table 2.

SIDE CHANNEL SPILLWAYS: HYDRAULIC THEORY, ECONOMIC FACTORS, AND EXPERIMENTAL DETERMINATION OF LOSSES

By Julian Hinds,* M. Am. Soc. C. E.

Synopsis

The design of a side channel spillway involves certain special hydraulic and economic factors which, it is thought, have not been previously discussed. On account of the disturbed conditions of flow, there are large internal frictional losses, the relative amounts of which are largely dependent on the magnitude and form of the individual installations. Flow can be computed by Bernoulli's theorem, using an experimentally determined coefficient of loss, similar to that of friction in ordinary flow, but such a coefficient would not be constant, and would need to be determined separately for many different sets of conditions. This situation can be avoided by utilizing the law of conservation of linear momentum for determining the flow. Formulas required for this purpose are developed.

The question of the proper location of the spillway on the ground is discussed, and a systematic method of determining the most economic design is suggested.

Although the fundamental principles utilized are well established in physics, the question arose as to the propriety of applying these principles to the conditions under discussion. A series of experiments was planned and carried out for the purpose of establishing the correctness of the assumption used. The results of experiments at the Arrowrock Spillway, and of laboratory experiments at Bellvue, Colo., are given in the paper.

The two most important conclusions reached are, as follows:

1.—Bernoulli's theorem is not conveniently applicable, because of lack of uniformity in the coefficient of loss.

2.—The law of the conservation of linear momentum is directly applicable without an experimental coefficient, subject only to a small correction for swell in volume due to entrained air and unequal distribution of velocities.

Introduction

The proper design of a spillway of the type shown in Fig. 1 involves a number of more or less novel problems, which, although simple if taken sepa-

Note.—The subject of this paper is one of ten selected by the Special Committee on Irrigation Hydraulics for study and research. The Committee, after studying the paper, has recommended its publication in the *Proceedings* in order to elicit discussion of the subject. (See Progress Report of the Committee, *Proceedings,* Am. Soc. C. E., March, 1925, Society Affairs, p. 137.)

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rately, form a complicated whole. The ultimate economy of construction depends on the form and dimensions of the spillway channel, its location, both in plan and elevation, its hydraulic properties, and other factors. The hydraulics presents a special problem, due to loss by shock and other irregularities caused by the constant addition to the flow in the channel of the water coming in over the spillway crest. Although this type of structure is very inefficient hydraulically, physical conditions sometimes make its use desirable. It is best suited to spillway discharges of moderate quantities. The determina-

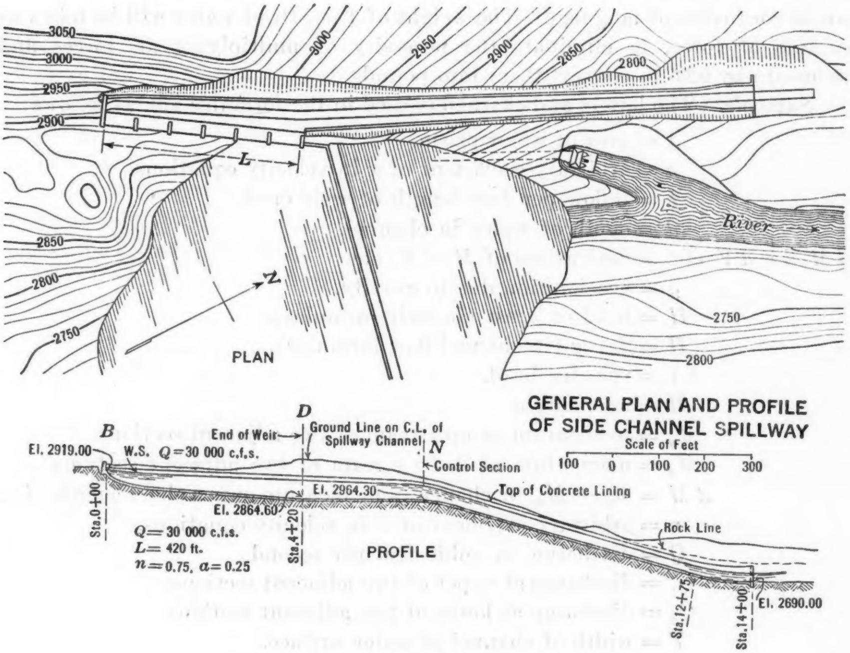


FIG. 1.

tion of the quantity of flood discharge to be provided for is beyond the scope of this paper. It will be assumed that the required spillway capacity is known. The depth of flow over the crest has an important influence on the cost of both the spillway and the dam. Generally, and within certain limits, the cost of the spillway channel is reduced by a short crest with a deep overflow, but if the spillway is of the open or uncontrolled type the cost of the dam will be increased, due to the excessive free-board required. If spillway gates are provided, their cost increases with their height. The factors involved in this part of the problem are too many and too varied to permit of a general solution. The most practical method of solving this problem is to assume a number of depths of overflow and prepare preliminary designs and cost estimates for the spillway, dam, and other related features.

Accuracy of Computations.—The quantity of flow to be provided for in the design of a spillway can be only roughly approximated at best, and extreme

precision in computation is not justified. As the preparation of this paper progressed, arithmetical simplifications were developed, and the method of procedure finally recommended differs from that used in some of the typical designs and the reduction of experimental data. However, all computations are based on the same fundamental theories, and sufficient checking has been done to show that the various methods of procedure yield results in sufficiently close agreement for all practical purposes.

Units of Measure.—Although the equations deduced are independent of any particular system of measurements, units of 1 ft. and 1 sec. will be used to avoid confusion of language. The weight of 1 cu. ft. of water will be taken as a unit of force, to eliminate the necessity of multiplying all forces and momenta by 62.5 to convert them into pounds.

Symbols.—The letters and characters used in the formulas are, as follows:

A = area of water prism.
a = arbitrary coefficient of x in velocity equations.
b = inflow per foot length of weir crest.
d = depth of water in channel.
dM, dx, dV, etc. = derivatives of M, x, V, etc.
g = acceleration due to gravity.
H = head on crest (in weir formulas).
$H = d + y$ (in channel-flow formulas).
hv = velocity head.
M = momentum.
M_u = momentum at up stream of two adjacent sections.
M_d = momentum at down stream of two adjacent sections.
$\Delta M = M_d - M_u$, or change in momentum over a short length, Δx.
n = arbitrary exponent of x in velocity equations.
Q = discharge, in cubic feet per second.
Q_1 = discharge at upper of two adjacent sections.
Q_2 = discharge at lower of two adjacent sections.
T = width of channel at water surface.
t = time, in seconds.
V = velocity, in feet per second.
V_1 = velocity at upper of two adjacent sections.
V_2 = velocity at lower of two adjacent sections.
$\Delta V = V_2 - V_1$, or change in velocity in a short distance, Δx.
x = distance along axis of channel.
Δx = distance between consecutive cross-sections of the channel.
y = ordinate to the water surface curve in the channel.

THE HYDRAULIC THEORY

Referring to Fig. 1 it will be noted that the flow moves over the crest in a direction approximately at right angles to the axis of the spillway channel. At the moment it comes in contact with the body of the water already in the channel, the incoming water has an appreciable velocity in a vertical plane normal to the channel axis. This transverse velocity is of no assistance in moving the water along the spillway channel.

The force producing axial motion in the channel is derived from the falling of the particles of water along the water surface curve. The useful fall of any particle is measured by the drop in the water surface below its elevation at the point at which the particle entered the stream. Fig. 2 represents diagrammatically a section along the center line of a side-channel spillway. The water surface is assumed to fall along some curve from the point, *B*, at the upper end of the channel to a point, *D*, opposite the down-stream end of the crest. Each particle coming into the channel contributes to the production of velocity and the overcoming of resistances between *B* and *D* an amount of energy represented by its effective fall, or the drop in water surface from the point at which it enters, to *D*. Thus, a particle coming in at *B* will have an effective fall equal to *P D*, whereas one coming in at *D* will have no effective fall. The total applied energy down to the point, *D*, is equal to that produced by the entire flow falling through the average drop for all the particles. If the surface curve from *B* to *D* is a straight line, and if the inflow per foot of crest is uniform, the average fall will be one-half the total fall, *P D*. If the curve is convexed, upward, as *B C D*, the average fall will be greater than $\frac{1}{2}$ *P D*, and if the curve is concaved, the proportion will be less than $\frac{1}{2}$ *P D*.

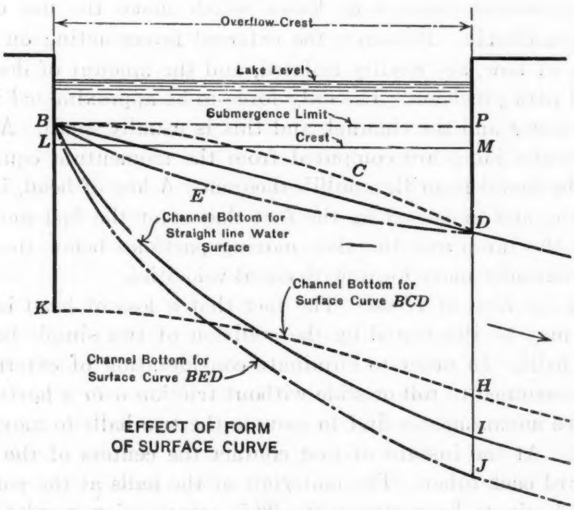


Fig. 2.

Only a part of the average fall is available for the production of velocity head at *D*. The remainder is used to overcome frictional and impact resistances. The frictional resistances are relatively small and may be neglected or estimated by methods used for other cases of variable flow. Impact is of greater importance.

Application of the Laws of Motion.—Water flowing in a channel must conform to each of the two laws of motion—the law of the conservation of energy and the law of the conservation of linear momentum. The first law is commonly expressed in hydraulics in the form of Bernoulli's theorem, which

states that the velocity head plus the static head at any point is equal to the velocity head plus the static head at any other point, plus or minus the intervening "losses." The word "loss" in this sense refers to the transformation of kinetic or static energy into some less available form, such as heat. The energy is not actually lost in the sense of being destroyed. The law of the conservation of linear momentum requires that the momentum of any system of particles can be changed only by the application of an external force, the amount of change being proportionate to the magnitude and duration of the force. This law is commonly expressed in the form: "Force is equal to mass times acceleration." In hydraulics, it is more convenient to make the equivalent statement that "force is equal to rate of change of momentum with respect to time."

Neither of these laws is inherently superior to the other, each having its particular field of usefulness. The results obtained from the proper application of the two laws always correspond. In special cases one or the other may be more readily applicable.

A good example of the special usefulness of the momentum theory is afforded in calculations for the hydraulic jump in a straight uniform channel, where violent surges cause heat losses which make the use of Bernoulli's theorem impracticable. However, the external forces acting on the water, in the direction of flow, are readily isolated, and the amount of deceleration can be computed with precision. The only force to be approximated is the friction between the water and the channel, and this is usually small. After flow conditions below the jump are computed from the momentum equation the loss of head can be found from Bernoulli's theorem. A loss of head, in such a case, is unavoidable, and is caused by the fact that after the fast moving particles coming into the jump and the slow moving particles below the jump collide they must eventually move away with equal velocities.

Necessity for Loss of Head.—The fact that a loss of head is necessary in such a case may be illustrated by the collision of two simple bodies, such as two billiard balls. In order to eliminate consideration of external forces the balls may be assumed to roll or slide without friction over a horizontal surface. It will also be advantageous first to assume the two balls to move in the same straight line. At the instant of first contact the centers of the two balls are moving toward each other. The materials of the balls at the point of contact immediately begin to be compressed. This compression results in a reaction which accelerates the forward ball and retards the rear ball. The compression increases until the velocities of the two balls are identical, at which time considerable energy has been expended in the compression of the balls. If the balls are perfectly elastic this energy is available, through the resilience of the balls, to force them apart again. If the balls are entirely inelastic no force will be available to separate them, and they will move together with identical velocities. As a result, there will be a reduction in the combined kinetic energy of the two balls equivalent to the elastic energy of the perfectly elastic balls at the instant of greatest compression. The important conclusion results that if all the energy of impact is absorbed the two balls will move away

together and, conversely, if the balls are to move away together all the energy of impact must be absorbed.

Momentum and Energy Laws Both Required.—If the elasticity is perfect, the relative velocity of the two balls after impact will be exactly the same as before impact, whereas if they are perfectly inelastic the resultant relative velocity will be zero. With intermediate degrees of elasticity the resultant relative velocity will be determined by the proportion of the energy of impact absorbed.

The individual action of the balls after impact may be determined only by the application of both the laws of conservation of energy and of momentum. There is only one combination of individual velocities that will satisfy both the energy and momentum requirements, and that combination is controlled by the energy absorbed in impact. That part of the energy of impact not absorbed is utilized in the production of relative velocity between the balls after impact, that is, the velocity of one ball with respect to the other. The average velocity can be determined from the equation of momentum, and if the relative velocity is known, the motion is completely determined. If the relative velocity is zero, the actual and average velocities are identical, and in this special case only, the factor representing the energy equation may be dispensed with, the law of the conservation of momentum affording a complete solution.

Impinging Streams of Water.—When two streams of water flowing at different velocities come together and flow away down a common conduit, the two systems of particles eventually intermingle and the particles move with approximately equal individual velocities. Consequently, the energy of impact must be lost, or changed into some form not readily available for the production of velocity. As in the simpler case of the impinging balls, flow for this special condition is completely determined from the momentum equation. This is because the momentum always precisely determines the average velocity, which, in this instance, is also the actual velocity.

The term "average velocity" should not be confused with the "mean velocity" as ordinarily used in hydraulic computations. Average velocity, as here used, is the arithmetical or algebraic average of the velocities of all the particles. The mean velocity as ordinarily used is the average velocity over the cross-section of the channel taken by areas. Unless the velocity is uniform over the entire cross-section of the channel at a given point, these two terms are not identical. Where the velocities vary, more particles pass through an area of high velocity than through an equal area of low velocity, and the average velocity is greater than the mean. The assumption that the velocity is uniform over a given cross-section introduces an error into all ordinary hydraulic computations, as this assumption is never strictly true. This factor is relatively of greater importance for side-channel spillways than for other more usual types of structures, because velocity variations are more pronounced. However, the relation of the actual and average velocities is too complicated to be given practical consideration, and the usual assumption of uniformly distributed velocities will be made.

The effect of this approximation on the hydraulics of impinging streams is interesting. As previously stated, the average velocity is completely and accurately determined from the momentum equation. If the velocity is uniform over the cross-section there will be no relative velocity between particles and all the energy of impact will be lost. If the velocity is not uniform over the section there will be some relative velocity, and a part of the energy of impact will not be lost. However, as shown, the average velocity is greater than the mean velocity, and water-prism areas and depths computed on the basis of the known average velocity will be too small. Thus, although for a given average velocity, the theoretical impact loss computed on the basis of a uniform distribution of velocities is greater than the actual impact loss, nevertheless, the theoretical depth computed on the same basis is smaller than the actual depth.

Hydraulic Formulas for Side-Channel Flow.—A condition of impact and shock loss exists at every point along a side-channel spillway from the beginning to the end of a spillway crest, as from B to D in Fig. 2. Flow in such a case is completely determined by making the momentum after impact equal to that before impact plus any acceleration due to external forces, subject only to the approximations mentioned previously. The same result may be obtained by placing the energy after impact equal to that before impact minus all the energy lost in impact. However, equating the momenta will be found to be more convenient.

Consider conditions at two consecutive sections, a small distance, Δx, apart, somewhere between B and D, Fig. 2. If the weir discharge per unit length is b, the inflow between the two sections will be $b \Delta x$. The velocity and the discharge at the up-stream section may be designated by V and Q, respectively, and at the down-stream section by $V + \Delta V$ and $Q + b \Delta x$. The momenta at the two sections, therefore, will be:

Up stream,

$$M_u = \frac{Q V}{g} \qquad (1)$$

Down stream,

$$M_d = \frac{Q + b \Delta x}{g} (V + \Delta V) \qquad (2)$$

Subtracting Equation (1) from Equation (2):

$$\Delta M = \frac{Q \Delta V}{g} + \frac{b \Delta x}{g} (V + \Delta V) \qquad (3)$$

As Δx approaches the limit zero, ΔV becomes infinitesimal as compared to V, and disappears, giving the differential equation:

$$\frac{d M}{d x} = \frac{Q}{g} \frac{d V}{d x} + V \frac{b}{g} \qquad (4)$$

The rate of change of momentum with respect to time being V times the rate of change with respect to x, we may write:

$$\frac{d M}{d t} = V \frac{Q}{g} \frac{d V}{d x} + \frac{b V^2}{g} \qquad (5)$$

The rate of change of momentum with respect to time is equal to the accelerating force, which in this case is the component of the weight of the water acting down the water-surface slope. If the discussion is confined to the horizontal part of the velocity and if x is taken as the horizontal distance along the channel, the accelerating force will be $Q \frac{d\,y}{d\,x}$, in which, $\frac{d\,y}{d\,x}$ is the tangent of the water-surface slope. Substituting this force for $\frac{d\,M}{d\,t}$ in Equation (5) and reducing, gives:

$$\frac{d\,y}{d\,x} = \frac{V}{g}\frac{d\,V}{d\,x} + \frac{b\,V^2}{Q\,g} \qquad (6)$$

Integrating,

$$y = \frac{1}{g}\int_0^x \left(V \frac{d\,V}{d\,x} + \frac{b}{Q}\,V^2\right) d\,x \qquad (7)$$

in which, y is the ordinate to the water-surface curve.

Special Formula for Simple New Designs.—If the relations of Q and V to x are known in a given case Equation (7) can be integrated and the form of the surface curve determined. These relations depend on the form and dimensions of the channel, and will be algebraically complicated unless the channel is purposely designed to make them simple. In preparing a new design there appears to be no objection to choosing a form of channel that will simplify the computations, or, what is the same thing, choosing the hydraulics and computing the dimensions of the structure to correspond.

The inflow per foot of spillway crest for the purpose of design will generally be uniform, and the total discharge at any section x distant from the upper end of the crest will be given by an equation of the form:

$$Q = bx \qquad (8)$$

An equation of the exponential type will be found convenient for expressing the velocity relation, and by properly choosing constants such an equation can be made to conform to a wide range of physical requirements. The following form is suggested:

$$V = ax^n \qquad (9)$$

In Equation (9), a and n are arbitrary constants and V and x denote, respectively, velocity of flow and distance from the upper end of the crest.

Substituting these values for V and Q in Equation (7), integrating and reducing:

$$y = \frac{a^2\,(n+1)}{2\,g\,n}\,x^{2n} \qquad (10)$$

Substituting $a^2\,x^{2n}$ for V^2, Equation (10) is simplified to:

$$y = \frac{n+1}{n}\,h\,v \qquad (11)$$

in which, $h\,v$ is the theoretical velocity head.

THE PREPARATION OF AN UNRESTRICTED DESIGN

Economic Factors.—A spillway channel is completely determined by Equations (8) to (11), if its shape and the values of a and n are chosen. The proper choice of these factors is controlled by economic considerations. The discussion of this part of the problem will be confined to a trapezoidel channel on a comparatively steep hillside, which is a usual case. The conclusions reached can be revised readily to suit other conditions.

The effect of the shape of the channel on the excavation is illustrated in Figs. 3 and 4. Safety usually demands that the channel be set well into the original formation. It may be required that the waterway be entirely in rock. If the water-surface elevation, channel side slopes, area of water prism, and location of point of outcrop, A, are fixed, it is evident from Fig. 3 that the excavation is reduced by a narrow bottom width of channel. It is similarly evident from Fig. 4 that, other things being constant, the side slopes should be made as steep as feasible. The minimum practical width of bottom will depend on the equipment to be used for removing the material from the trench. If the excavation is to be done by machinery, a width of 15 or 20 ft. may be required. For team work a somewhat narrower base may be used. The reduction in excavation for extremely narrow widths is not great. The side slopes should be trimmed to the steepest angle at which the materials will safely stand.

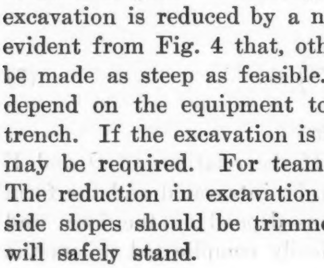


FIG. 3.

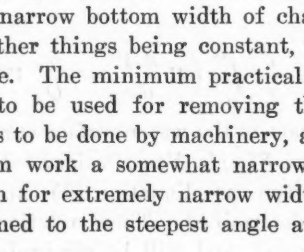


FIG. 4.

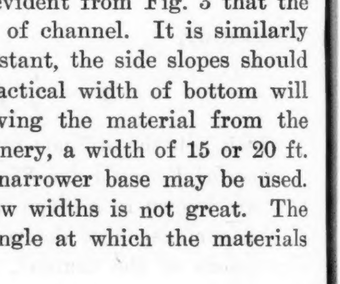


FIG. 5.

In many cases it will be necessary to line the spillway channel with concrete. Other things being constant, the cost of lining, which is an important item, is least when the bottom width is such that the wetted perimeter is a minimum. With steep side slopes this will require an average width of water prism equal approximately to twice the depth of the water. The bottom of the channel may be made somewhat narrower without greatly increasing the amount of lining, but the cost of lining should be considered in the final selection of channel width.

Figs. 3 and 4 are drawn to represent that part of the channel down stream from the crest structure, but the same principles apply to the part of the channel opposite the crest.

With a movable crest of the drum type an additional factor is introduced, due to the necessity for supporting the crest on a concrete base. It will be found advantageous to set the channel back into the hill a certain distance to reduce the quantity of concrete required for this purpose. The distance, z,

in Fig. 5, should be chosen to make the combined cost of the shaded part of the concrete and the shaded part of the excavation a minimum. After a tentative bottom profile has been determined, the selection of which will be discussed later, the correct value of z can be calculated for a number of cross-sections, and a theoretical plan of the structure plotted on a contour map of the site. Unless the topography is unusually regular, it will not be possible to fit a practicable structure to the computed points with accuracy, but a location approximating the computed locations may be selected for trial.

After the cross-section of the channel has been selected, the profile is controlled by the values of a and n. Assuming a specified drop in the water surface from B to D, the effect of varying n is illustrated roughly in Fig. 2. If $n = 0.5$ the surface curve will be straight and the channel bottom will follow some such line as $B\,I$. If n is greater than 0.5 the water-surface curve will be convexed upward, as $B\,C\,D$. If n is exactly 1.0 the surface curve will be a parabola and the bottom will be a parallel curve, as $K\,H$. If n lies between 0.5 and 1.0 the bottom line will start at B, but if the value of n is nearly unity, it will drop rapidly nearly to the line, $K\,H$, which line it will cross up stream from H. If n exceeds unity, the bottom line theoretically drops to an infinite depth at the upper end of the channel, then rises rapidly approximately to the line, $K\,H$, which it crosses before reaching H. These statements can be readily proved by making a few trial computations from hypothetical data.

The effect on the profile, of varying both n and a, is shown in Fig. 6. If the value of n is known or assumed, the greatest economy in excavation at any point is obtained by making a such that the total drop from the submergence limit at B to the bottom of the channel at the point considered, expressed as $y + d$, is a minimum. Choosing H to represent $y + d$, the minimum value can be found by an equation, derived as follows:

$$H = d + y \qquad (12)$$

$$= d + \left(\frac{n+1}{n}\right) h\,v \text{ (from Equation (11))} \qquad (13)$$

As $h\,v$ is equal to $\frac{V^2}{2\,g}$ and V^2 is equal to $\frac{Q^2}{A^2}$:

$$H = d + \left(\frac{n+1}{n}\right) \frac{V^2}{2\,g}$$

$$= d + \left(\frac{n+1}{n}\right) \frac{Q^2}{2\,g\,A^2} \qquad (14)$$

Differentiating with respect to d:

$$\frac{d\,H}{d\,d} = 1 - \left(\frac{n+1}{n}\right) \frac{Q^2}{g\,A^3} \frac{d\,A}{d\,d} \qquad (15)$$

Making $\frac{d\,H}{d\,d}$ equal to zero, the minimum value of H is found to obtain when:

$$g\,A^3 = \left(\frac{n+1}{n}\right) Q^2 \frac{d\,A}{d\,d} \qquad (16)$$

The factor, $\frac{d\,A}{d\,d}$, is equal to T, the top width of the water prism. Inserting this value and recalling that $\frac{Q^2}{2\,g\,A^2} = h\,v$, Equation (16) simplifies to:

$$h\,v = \frac{n}{n+1}\,\frac{A}{2\,T} \qquad (17)$$

or, from Equation (11):

$$y = \frac{A}{2\,T} \qquad (18)$$

Having chosen a value for either a or n, the corresponding value of the other required to give the greatest economy at a specified point in a given channel, can be found from these equations.

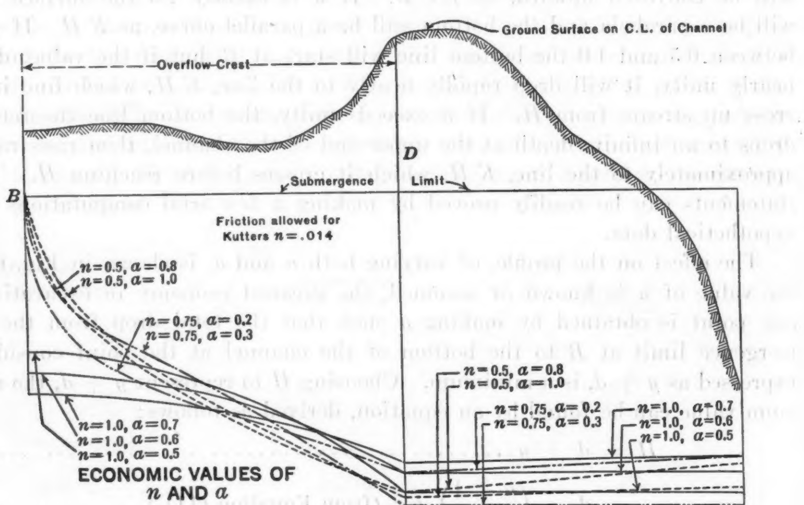


FIG. 6.

The correct combination of values for a and n to give the greatest economy for the entire structure is not so readily found. A general solution involves the equation of the surface of the hillside which can not be practically expressed algebraically. The problem is best solved by trial.

Anticipating the results of the study, it may be stated that n is a much more constant factor than a, and is more easily guessed. A start may be made, therefore, with an assumed value of n. Having selected the trial value, choose some station along the channel where the excavation is unavoidably heavy and assume that this section will be a controlling factor in the cost of the spillway. Usually, as a safety precaution, the channel must be set well into the hill opposite the end of the dam, with the result that the excavation is heavy near the point, D, Fig. 6. Below some point, N, the channel need be set into the rock only as far as may be required to afford secure sidewalls. The section at D, therefore, influences the economy of construction

more than any other single section, and the maximum economy of excavation for a given value of n, will be approximated if $d + y$ is made a minimum at D.

Therefore, unless special conditions indicate that the controlling section is elsewhere, find the economic value of y for the point, D, from Equations (17) and (18) by trial* and the corresponding value of a from Equation (10). The velocity and the water-surface curve may then be found from Equations (9) and (10), or Equation (11). Knowing the velocity and the form of the channel, the depth of flow and the bottom profile at points above D are readily obtained. Below D it is economical to have the bottom of the channel as high as possible without interfering with flow above D. Below N (Figs. 1 and 6), the channel will usually be steep and flow will occur at a velocity in excess of the critical velocity.

The velocity at D, determined as outlined, will be less than the critical. A "control", or point of critical flow, therefore, may be provided at N or between D and N. The control may be anywhere within this reach, but should be a sufficient distance below D to permit the necessary change in velocity to take place gradually. In the examples illustrated, the control is assumed to be at N. Having located the control, the hydraulic computations may be completed and the profile platted down stream from B to the point, N. The grade from D to N may be level, or even reversed.

Comparative Estimates.—As soon as a bottom profile has been completed the principles of Fig. 5 may be applied to several sections of the spillway crest, and the resulting channel platted on a topographic map of the site. A cost estimate for the spillway may then be prepared, which will locate one point on one of the curves in the left-hand part, (a), of Fig. 7. The correctness of the first trial value of a should be checked by assuming additional trial values, with the same value of n, and making more cost estimates. The result will give a number of points from which one of the curves in Fig. 7 (a) may be drawn. The lowest point on this curve will give the correct value of a for use with the assumed value of n. A new value of n should then be assumed, and the process repeated, platting other curves as shown in the diagram, until it is clear that the most suitable combination of a and n has been found. The curve in Fig. 7 (b) is obtained by platting the low points of the curves in Fig. 7 (a) against their respective values of n.

In the example given it is seen that 0.75 is the best value for n, the corresponding value for a being approximately 0.25. This value of a is somewhat larger than the value corresponding to the value of y computed for least depth at D. In all usual cases, with a lined channel, this will be true, and trial values of a should never be less than required to give the minimum value of H at D unless the estimates show smaller values to be necessary.

Submergence of Crest.—An important matter not yet discussed is the amount the weir can be submerged at B without seriously reducing the dis-

* Assume a trial depth, d, and compute the area, A, and the top width, T. From Equation (17), find h_v and the corresponding velocity, V. Then, if $A V$ is equal to the required discharge, the assumed depth is correct. Otherwise, assume a new depth and repeat the process, continuing until an agreement is secured. Having thus arrived at the economic depth, the corresponding value of y is found from Equation (18).

charge. It has been the general practice heretofore to assume that Herschel's coefficients for ordinary submerged weirs are approximately applicable. The accuracy of this assumption will be discussed subsequently in connection with the experimental data. The economic importance of this point is apparent from the fact that the bottom profile is obtained by measuring $d + y$ downward from the permissible submergence line. In Figs. 1 and 6 the submergence is assumed to be such that the inflow will not be reduced, that is, according to Herschel, about 0.13 of the head on the crest. This small allowable submergence is chosen in these examples principally to simplify the theoretical discussion by making the inflow per foot constant. According to experiments, submergence reduces the discharge only slowly up to a fairly considerable depth. Furthermore, in a spillway the back-water falls away below the point, B. The crest may be submerged at B by one-half or two-thirds of the head without seriously reducing the total discharge. Equation (8) and Equations (10) to (18), inclusive, are not strictly applicable to such a partly submerged condition, as the inflow is variable. However, they apply with sufficient accuracy to any ordinary case, and the results may be checked by methods to be given subsequently. It may be desirable sometimes to assume a heavy submergence, increasing the depth of spill to compensate for the reduced discharge factor. The limit to which this process may be carried can be determined by cost estimates.

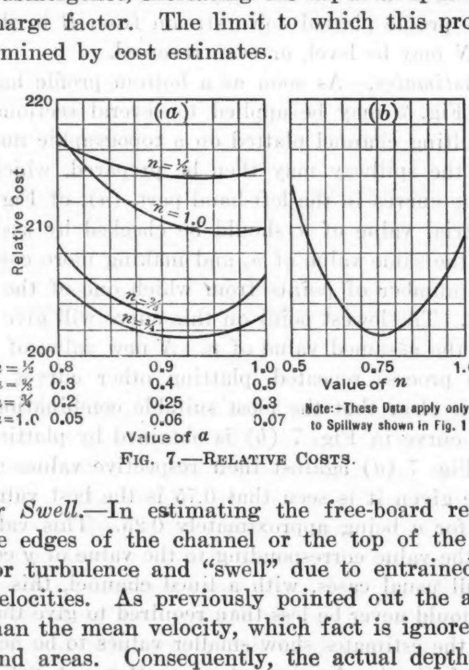


FIG. 7.—RELATIVE COSTS.

Allowance for Swell.—In estimating the free-board required to prevent slopping over the edges of the channel or the top of the lining, allowance must be made for turbulence and "swell" due to entrained air and unequal distribution of velocities. As previously pointed out the average velocity is always greater than the mean velocity, which fact is ignored in the computation of depths and areas. Consequently, the actual depth may be expected to exceed the computed depth by an amount depending on the extent of longitudinal eddies. The volume will be further increased by air drawn into the stream by the infalling water. It is believed that the only information available on this subject is that given subsequently in connection with the experimental data. The average bulking observed for a small model was about 4%

with a variation from zero to about 10 per cent. Observations on full-sized structures under normal flow conditions are needed.

Computations for a Restricted Design

Hydraulic Formulas.—The information already given is intended for use in the preparation of new designs not hampered by special limiting conditions. The same fundamental hydraulic principles may be applied to the analysis of an existing channel, or to the design of a channel which must conform to certain prescribed dimensions and grades, or where inflow is non-uniform due to excessive submergence or other cause. Equation (6) can be rewritten to make it applicable, approximately, to finite values of Δx. Equation (3) divided by Δx gives:

$$\frac{\Delta M}{\Delta x} = \frac{Q \Delta V}{g \Delta x} + \frac{b}{g}(V + \Delta V) \ldots\ldots\ldots\ldots\ldots (19)$$

Multiplying by the average velocity, $V + \frac{1}{2} \Delta V$,

$$\frac{\Delta M}{\Delta t} = \frac{Q}{g}\left(V + \frac{1}{2} \Delta V\right) \frac{\Delta V}{\Delta x} + \frac{b}{g}(V + \Delta V)\left(V + \frac{1}{2} \Delta V\right) \ldots (20)$$

As $\frac{\Delta M}{\Delta t}$ = accelerating force = $\frac{\Delta y}{\Delta x}$ times average discharge, Equation (20) becomes:

$$\frac{\Delta y}{\Delta x}\left(Q + \frac{1}{2} \Delta Q\right) = \frac{Q}{g}\left(V + \frac{1}{2} \Delta V\right) \frac{\Delta V}{\Delta x} + \frac{b}{g}(V + \Delta V)\left(V + \frac{1}{2} \Delta V\right) \ldots (21)$$

From which,

$$\Delta y = \frac{Q}{g}\left(\frac{V + \frac{1}{2} \Delta V}{Q + \frac{1}{2} \Delta Q}\right)\left[\frac{\Delta V}{\Delta x} + \frac{b}{Q}(V + \Delta V)\right] \Delta x \ldots\ldots\ldots (22)$$

or:

$$\Delta y = \frac{Q_1}{g} \frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left[\Delta V + \frac{b V_2 \Delta x}{Q_1}\right] \ldots\ldots\ldots\ldots\ldots (23)$$

or:

$$\Delta y = \frac{Q_2}{g} \frac{(V_1 + V_2)}{(Q_1 + Q_2)}\left[\Delta V + \frac{b V_1 \Delta x}{Q_2}\right] \ldots\ldots\ldots\ldots\ldots (24)$$

in which, Q_1 and V_1 are the discharge and velocity at the up-stream end of the reach, and Q_2 and V_2 are the same functions at the down-stream end. It is interesting to note that when b is zero, $Q_1 = Q_2$ and Equations (22), (23), and (24) reduce to $\Delta y = \frac{V_2^2 - V_1^2}{2g}$, which is the energy equation for ordinary variable flow, neglecting friction. Equations (22), (23), and (24) are identical, except as to form, and may be used interchangeably, as found convenient.

Location of Control.—These equations are not as cumbersome as might at first appear, but before they can be applied, a starting place, at which the

velocity is known, must be found. Such a starting place will be located at a point of control, where the depth passes through the critical point, from above to below.

If the channel slope between D and N, Fig. 1, is insufficient to support flow at the critical depth, and below N, more than sufficient, the control will be located at N, and the computation of the hydraulics may be started from that point. If the slope between D and N is greater than is required to support flow at the critical depth, the control will come at D, or at some point above. Up stream from D there is, in addition to the force of friction, a resisting force due to impact. This force has the same effect on the formation of a control as a flattening out of the grade. If the slope of the channel is insufficient to overcome losses and maintain flow at the critical depth immediately above D, then the control will be at D. If the slope immediately above D is more than sufficient for maintaining critical flow, then the control will come at some point farther up stream, where the slope becomes insufficient for this purpose.

The actual location of a control above D is complicated by the fact that the critical depth, the impact resistance slope for critical depth, and the discharge are all variable. If the equation of the critical velocity can be written,

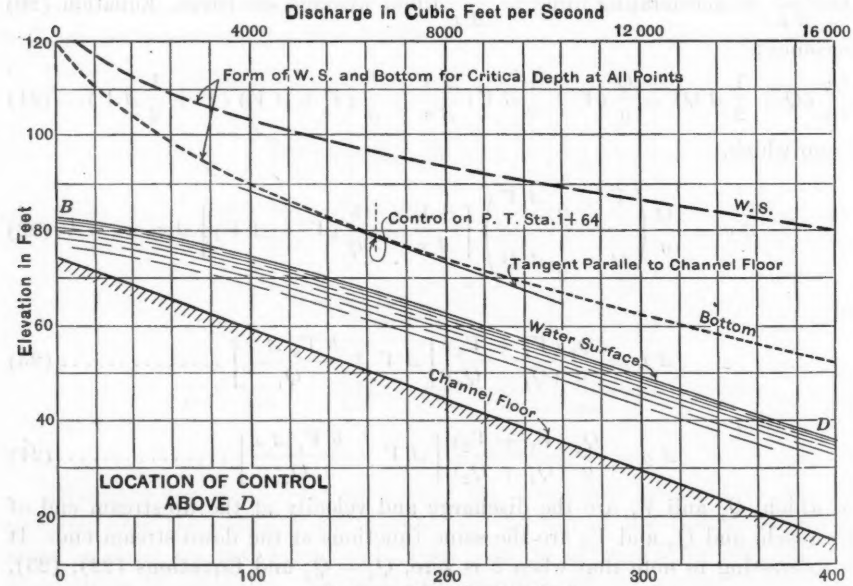


FIG. 8.

the slope can be found from Equation (6), but this can seldom be done. A suggested method of attacking the problem is illustrated in Fig. 8 and Tables 1, 2, and 3. First, compute the critical velocities and discharges corresponding to a number of depths, as shown in Table 1. The hydraulic radii may also be computed and recorded for use in estimating friction losses. From

Table 1 values of the critical depths and velocities may be taken, by interpolation, for use in Table 2, which is self-explanatory and gives in Column (13) the drop in water surface necessary to maintain flow at the critical depth throughout the full length of the channel. Starting from an arbitrary water-surface elevation at some station, a profile of the channel, for critical flow at all points, can be platted, as shown by dashed lines in Fig. 8. A tangent parallel to the bottom of the actual channel may then be drawn to the

TABLE 1.—Computations for Critical Depths. For Channel in Fig. 8.

(Bottom width, 10 ft.; side slopes, $\frac{1}{2}$: 1)

Depth, in feet, d.	Area, in square feet, A.	Top width, in feet, T.	Velocity head, $\frac{A}{2T}$.	Critical velocity, in feet per second.	Discharge, Q.	Hydraulic radius, in feet.
2	22	12	0.92	7.68	169	1.52
4	48	14	1.71	10.49	504	2.53
6	78	16	2.44	12.52	978	3.33
8	112	18	3.11	14.15	1 585	4.01
10	150	20	3.75	15.53	2 330	4.63
12	192	22	4.36	16.75	3 216	5.22
14	238	24	4.96	17.86	4 252	5.76
16	288	26	5.54	18.88	5 440	6.29
18	342	28	6.11	19.82	6 780	6.82
20	400	30	6.67	20.71	8 284	7.31
22	462	32	7.22	21.55	9 960	7.81
24	528	34	7.76	22.34	11 800	8.29
26	598	36	8.31	23.12	13 820	8.77
28	672	38	8.84	23.84	16 020	9.26

TABLE 2.—Computations for Locating Control. For Channel in Fig. 8.

(Bottom width, 10 ft.; side slopes, $\frac{1}{2}$: 1.)

$$\Delta y = \frac{Q_1 (V_1 + V_2)}{g \; (Q_1 + Q_2)} \left\{ \Delta V + \frac{b \; V_2 \, \Delta x}{Q_1} \right\}$$

x	Δx	Q	$Q_1 + Q_2$	d_c	V_c	$V_1 + V_2$	ΔV	$\frac{b V_2 \Delta x}{Q_1}$	$\Delta V + \frac{b V_2 + \Delta x}{Q_1}$	Δy	h_f*	$\Delta y + h_f$
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
0	......	......	......	......	......	......	......	......	......	......	......	......
10	10	400	400	3.4	10.0	10.0	10.0	....	....	.....	0.03	....
25	15	1 000	1 400	6.2	12.5	22.5	2.5	18.8	21.3	4.25	0.04	4.29
50	25	2 000	3 000	9.2	14.9	27.4	2.4	14.9	17.3	4.91	0.08	4.99
100	50	4 000	6 000	13.5	17.6	32.5	2.7	17.6	19.3	6.77	0.15	6.92
150	50	6 000	10 000	16.9	19.3	36.9	1.7	9.6	11.3	5.21	0.15	5.36
200	50	8 000	14 000	19.7	20.6	39.9	1.3	6.8	8.1	4.33	0.15	4.48
250	50	10 000	18 000	22.1	21.6	42.2	1.0	5.4	6.4	3.74	0.15	3.89
300	50	12 000	22 000	24.2	22.4	44.0	0.8	4.5	5.3	3.30	0.15	3.45
350	50	14 000	26 000	26.2	23.2	45.6	0.8	3.8	4.6	3.04	0.15	3.19
400	50	16 000	30 000	28.0	23.8	47.0	0.6	3.4	4.0	2.73	0.15	2.88
......	......	......	......	......	......	......	......	......	......	38.28	1.17	39.45

* h_f = friction loss, computation not shown.

resulting bottom line of the critical depth channel, and the point of control will be located at the point of tangency. Referring to Fig. 8, it is evident that the slope required to maintain critical flow to the left of Station 1 + 64 is greater than the actual slope, and to the right it is less, which is the condition necessary to the formation of a control. If more than one point of tangency is possible, that one giving the lowest position of the tangent will be likely to control. It is possible to have two or more control points with hydraulic jumps between.

The Back-Water Curve.—Having located a control, the back-water curve may be calculated both ways, as illustrated in Table 3, and as platted in the lower part of Fig. 8. Equation (23) is used in the upper part of Table 3 and Equation (24) in the lower part, in order to take advantage of as many constant terms as possible. The accuracy of the computations will depend on the length and number of subdivisions assumed. The number of divisions used in the examples shown could be increased to advantage.

Experiments Upon a Model Structure at Bellvue, Colo.

In the latter part of December, 1923, the U. S. Bureau of Reclamation constructed a model spillway at the Bellvue Laboratory of the U. S. Department of Agriculture, near Fort Collins, Colo., and conducted a series of experiments for the purpose of establishing, if possible, the correctness of the foregoing theory. The experiments were made by the writer under the direction of J. L. Savage, M. Am. Soc. C. E., Designing Engineer of the Bureau of Reclamation. A large part of the observational work was done by Ivan E. Houk, M. Am. Soc. C. E., and Samuel Judd, Engineer, Bureau of Reclamation. Valuable assistance in operating the laboratory was given by R. L. Parshall, Affiliate Am. Soc. C. E., Irrigation Engineer, U. S. Department of Agriculture.

The general arrangement and principal dimensions of the Bellvue Laboratory and the model spillway are shown in Fig. 9. The intake to the flume is adjacent to the head-works of the Jackson Ditch, and is controlled by a single 4 by 4-ft. sliding gate. Two 2 by 2-ft. waste-gates are available for additional control, and an 8-in. pipe line, with a gate valve near the weir pool, may be utilized for the final adjustment of flow.

A 10-ft. Francis weir for measuring the discharge and baffles for stilling the flow are located as shown. Fig. 10 is a general view of the laboratory and Fig. 11 shows the spillway structure.

The spillway crest was of sheet iron rounded to a 9-in. radius, and was approximately 16 ft. long. The trough was of 1-in. planed boards carefully joined on the edges and made water-tight with fiber cement. The leakage was negligible.

That part of the structure carrying the rounded crest, corresponding to the part, *B D,* of Fig. 1, was located above a tight bulkhead across the test flume. In order to simulate conditions from *D* to *N,* Fig. 1, a short length of channel was provided below the end of the spillway crest. This part of the channel was located below the tight bulkhead. A second stop-plank bulkhead below the end of the channel was available for controlling the submerg-

TABLE 3.—COMPUTATIONS FOR BACK-WATER CURVE.

FOR CHANNEL IN FIG. 8.

(Bottom width, 10 ft.; side slopes, $\frac{1}{2}$: 1.)

x.	Δx.	Bottom elevation.	Trial Δy.	Surface elevation.	Depth, d.	Area, A.	Discharge, Q.	Velocity, V.	$Q_1 + Q_2$.	$\frac{Q_1}{g\,(Q_1 + Q_2)}$.	$V_1 + V_2$.	ΔV.	$\frac{b.\,V_2\,\Delta x}{Q_1}$	$\Delta V + \frac{b\,V_2\,\Delta x}{Q_1}$	Δy_m *	h_f.	$\Delta y_m + h_f$	Error.	Notes.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
164		49.10	....	66.80	17.70	333.6	6 560	19.70	......	......	......	....	......	......	....	....	....	....	......
100	64	58.70	7.90	74.70	16.00	288.0	4 000	13.89	10 560	0.0118	33.59	5.81	12.61	18.42	7.29	0.15	7.44	0.46	
			7.10	73.90	15.20	267.5	.....	14.95	......	......	34.65	4.75	.....	17.36	7.09	....	7.24	0.14	
			7.27	74.07	15.37	271.8	.....	14.72	......	......	34.42	4.98	.....	17.59	7.13	....	7.28	0.01	O. K.
50	50	66.20	5.00	79.07	12.87	211.5	2 000	9.46	6 000	0.0104	24.18	5.26	14.72	19.98	5.02	0.07	5.09	0.09	
			5.08	79.15	12.95	213.5	.....	9.37	......	......	24.09	5.35	.....	20.07	5.01	....	5.08	0.00	O. K.
25	25	69.95	1.80	80.95	11.00	170.5	1 000	5.86	3 000	0.0104	15.23	3.51	9.37	12.88	2.03	0.02	2.05	0.25	
			2.06	81.21	11.26	176.0	.....	5.68	......	......	15.05	3.69	.....	13.06	2.04	...	2.06	0.01	O. K.
10	15	72.20	0.59	81.80	9.60	142.1	400	2.82	1 400	0.0089	8.50	2.86	8.52	11.38	0.86	0.01	0.87	0.29	
			0.87	82.08	9.88	147.6	.....	2.71	......	......	8.39	2.97	.....	11.49	0.86	....	0.87	0.00	O. K.
0	10	73.70	0.23	82.31	8.61	Δy assumed = $2\,h_v$ at $x = 10$													

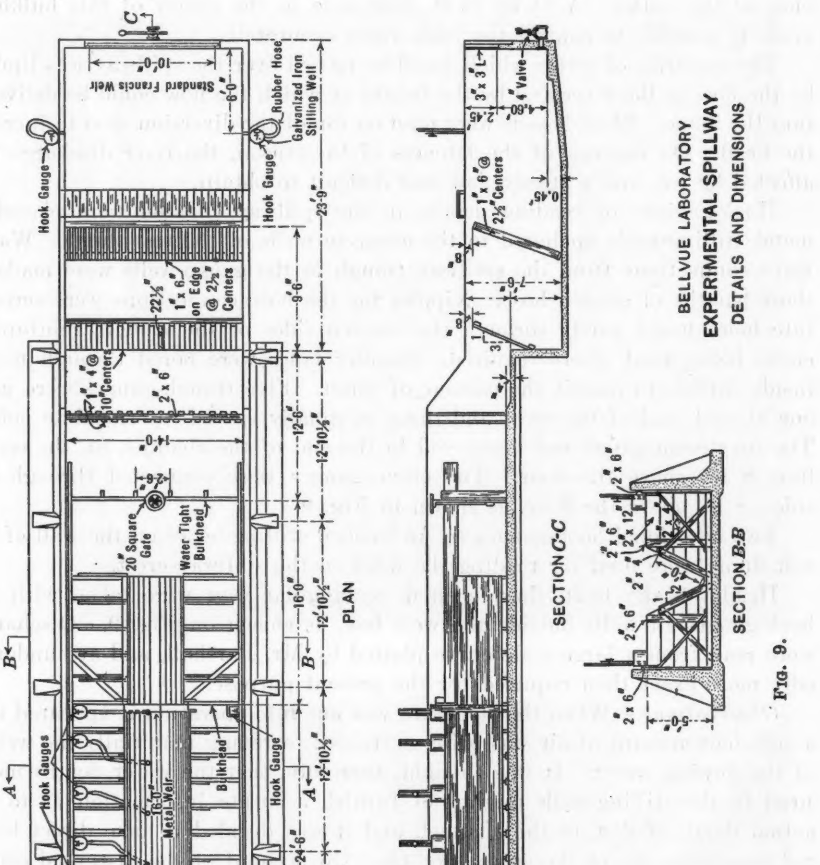
$$^*\Delta y = \frac{Q_1\,(V_1 + V_2)}{g\,(Q_1 + Q_2)}\left\{\Delta V + \frac{b\,V_2\,\Delta X}{Q_1}\right\} = \text{Column (11)} \times \text{Column (12)} \times \text{Column (15)} \quad \ldots\ldots(24)$$

TABLE 3.—(*Continued.*)

x.	Δx.	Bottom elevation.	Trial Δy.	Surface elevation.	Depth, d.	Area, A.	Discharge, Q.	Velocity, V.	$Q_1 + Q_2$.	$\frac{Q_2}{g(Q_1+Q_2)}$.	$V_1 + V_2$.	ΔV.	$\frac{b\,V_1\,\Delta x}{Q_2}$	$\Delta V + \frac{b\,V_1\,\Delta x}{Q_2}$	Δy_m*	h_f†	$\Delta y_m + h_f$	Error.	Notes.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)
164		49.10	....	66.80	17.70	333.6	6 560	19.70	......	......	.....	....	.....	.....	....	....	....	....	......
200	36	43.70	4.10	62.70	19.00	370.5	8 000	21.59	14 560	0.0171	41.29	1.89	3.55	5.44	3.82	0.12	3.94	0.16	
			4.50	62.30	18.60	359.0	......	22.27	......	......	41.97	2.57	.....	6.12	4.40	....	4.52	0.02	
			4.45	62.35	18.65	360.4	......	22.20	......	......	41.90	2.50	.....	6.05	4.34	....	4.46	0.01	O. K.
250	50	36.20	6.15	56.20	20.00	400.0	10 000	25.00	18 000	0.0173	47.20	2.80	4.44	7.24	5.92	0.18	6.10	0.05	
			6.22	56.13	19.93	397.9	......	25.13	......	......	47.33	2.93	.....	7.37	6.03	....	6.21	0.01	O. K.
300	50	28.70	6.63	49.50	20.80	424.3	12 000	28.29	22 000	0.0170	53.42	3.16	4.19	7.35	6.68	0.24	6.92	0.29	
			6.40	49.73	21.03	431.4	......	27.81	......	......	52.94	2.68	.....	6.87	6.18	....	6.42	0.02	
			6.39	49.74	21.04	431.7	......	27.79	......	......	52.92	2.66	.....	6.85	6.16	....	6.40	0.01	O. K.
350	50	21.20	6.51	43.23	22.03	463.0	14 000	30.24	26 000	0.0138	58.03	2.45	3.97	6.42	6.24	0.30	6.54	0.03	
			6.48	43.26	22.06	463.9	......	30.18	......	......	57.97	2.39	.....	6.36	6.18	....	6.48	0.00	O. K.
400	50	13.50	6.73	36.53	23.03	495.5	16 000	32.29	30 000	0.0166	62.47	2.11	3.78	5.89	6.10	0.35	6.45	0.28	
			6.92	36.34	22.84	489.2	......	32.71	......	......	62.89	2.53	.....	6.31	6.58	....	6.93	0.01	O. K.

$$* \Delta y = \frac{Q_2\,(V_1 + V_2)}{g\,(Q_1 + Q_2)} \left\{ \Delta V + \frac{b\,V_1\,\Delta X}{Q_2} \right\} = \text{Column (11)} \times \text{Column (12)} \times \text{Column (15)} \ldots\ldots (25)$$

† h_f = friction head, computations not shown.



BELLVUE LABORATORY
EXPERIMENTAL SPILLWAY
DETAILS AND DIMENSIONS

FIG. 9.

ence at the outlet. A $2\frac{1}{2}$ by $2\frac{1}{2}$-ft. slide-gate in the center of this bulkhead made it possible to control the back-water accurately.

The quantity of water which could be passed over the spillway was limited by the flow in the river and by the height at which the flow could be delivered into the flume. Flash-boards were used on top of the diversion dam to increase the head. On account of the lateness of the season, the river discharge was affected by ice, and a steady flow was difficult to obtain.

Hook-gauges for reading depths in the spillway channel were placed in metal stilling-wells anchored to the concrete walls of the test flume. Water-tight connections from the spillway trough to the gauge-wells were made by short lengths of garden hose. Nipples for the hose connections were screwed into holes bored partly through the wooden sides of the trough, reinforcing cleats being used where required. Smaller holes were bored through to the inside surface to permit the passage of water. Five trough-gauges were used, one at each end of the crest and three at equally spaced intermediate points. The up-stream gauge was connected to the end of the channel, on the center line, 8 in. above the floor. The other gauges were connected through the sides, 8 in. above the floor, as shown in Fig. 9.

Two additional hook-gauges set in wooden stilling-boxes on the wall of the test flume were used for reading the head on the spillway crest.

Heads on the 10-ft. sharp-crested, rectangular weir were taken with two hook-gauges in wells outside the weir box, as shown on Fig. 9. Discharges were read from a large-scale curve platted by Mr. Parshall, and are undoubtedly more exact than required for the present purpose.

Observations.—When the structure was put into operation it appeared that a sufficient amount of air was being entrained to reduce materially the weight of the flowing water. It was thought, therefore, that the water depths measured in the stilling-wells might not furnish accurate information as to the actual depth of flow in the channel, and it was decided to take direct level-rod measurements of the surface profile. On account of the disturbed condition of flow, it was not expected that these readings would be accurate, but later they were found to be consistent and reliable.

The conditions shown in Figs. 12 and 13 and Figs. 14 and 15 are typical, respectively, for non-submerged and submerged flows. The level-rod was held opposite each gauge connection and at such elevation that, as far as could be judged, it was in and out of the water equal intervals of time. Generally, two readings were taken for each station, one on top of the boil shown in the photographs, and one in the depression on the crest side of the channel. In some cases, where the flow was less regular, extra readings were taken near the middle of each section. Level-rod readings were also depended on for determining the back-water level in the pool into which the channel discharged.

The program for making the observations was arranged to eliminate, as far as practicable, personal equation and chance. Tests with varying discharges were run on three settings of the trough, two with a flat bottom slope and one with a considerable grade, as shown on the profiles, Figs. 16, 17, and 18. Fifteen separate runs were made. Preparation for each run was made by setting the head-gates and by-passes to approximate the flow required.

FIG. 10.—VIEW OF BELLVUE, COLO., LABORATORY.

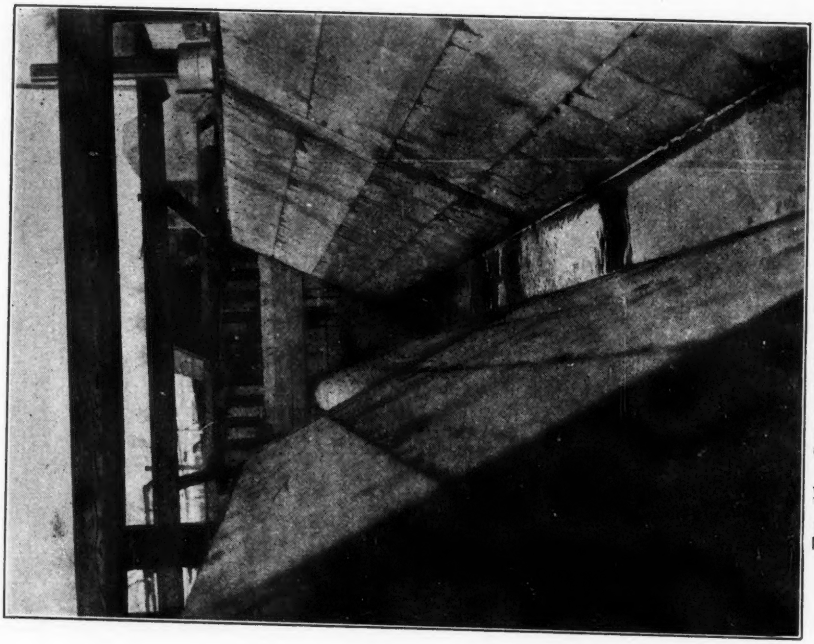


FIG. 11.—SPILLWAY STRUCTURE, SETTING NO. 1.

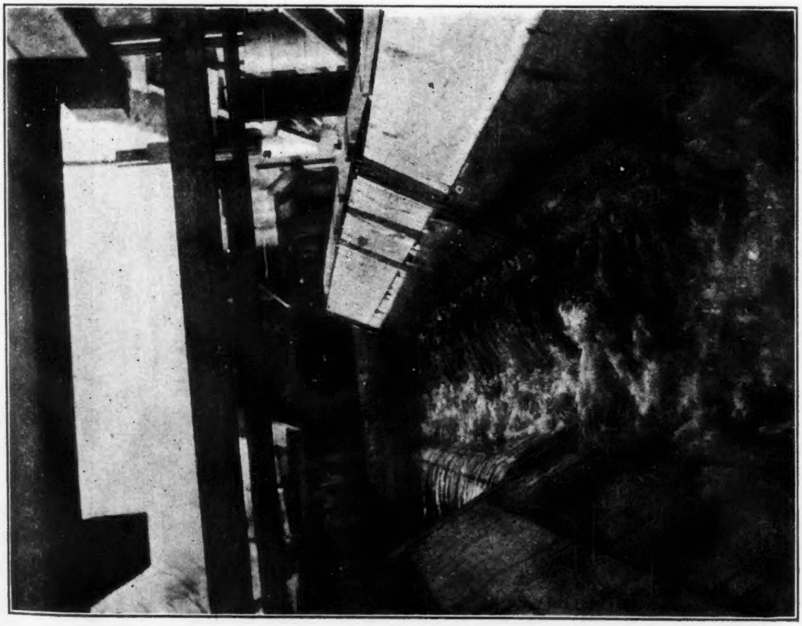


FIG. 12.—RUN NO. 1. DISCHARGE, 8.35 SEC-FT. TYPICAL FOR NON-SUBMERGED FLOW.

FIG. 13.—RUN NO. 6. DISCHARGE, 22.85 SEC-FT. NON-SUBMERGED FLOW.

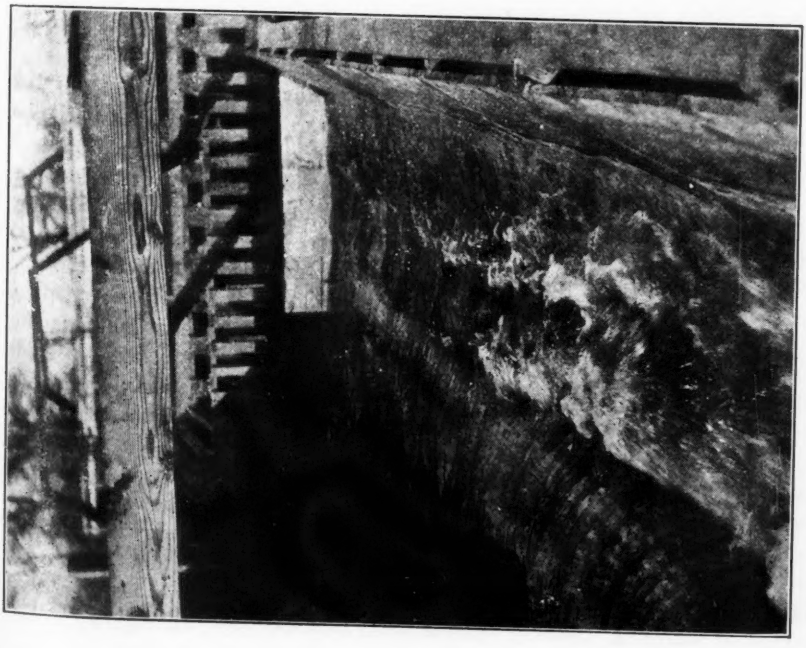


FIG. 14.—RUN NO. 3. DISCHARGE, 13.89 SEC-FT. TYPICAL FOR SUBMERGED FLOW.

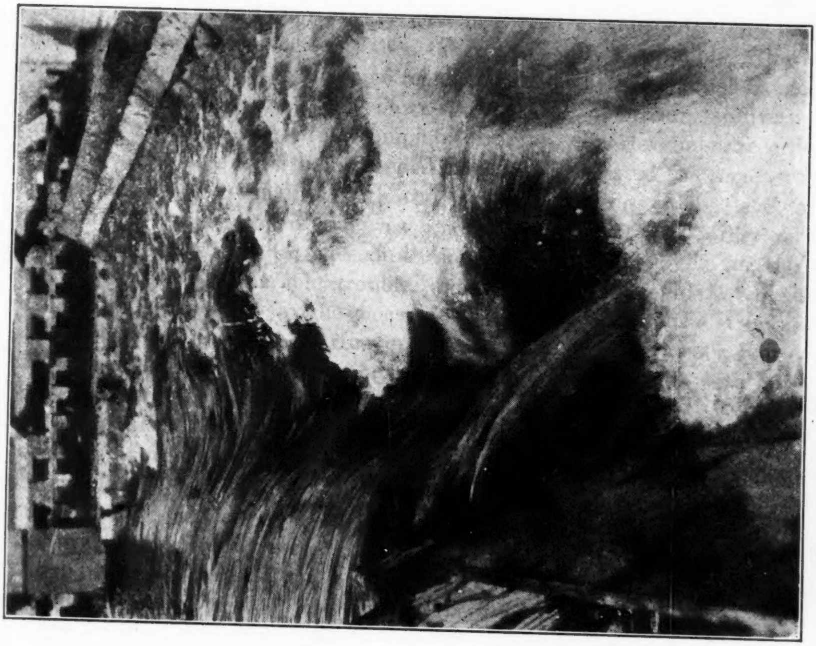


FIG. 15.—RUN NO. 7. DISCHARGE, 27.19 SEC-FT. SUBMERGED FLOW.

As soon as the depth in the gauge-boxes at the Francis weir became constant, it was assumed that steady flow had been established. The level-rod profile of the water surface was then taken as rapidly as possible, after which the rodman beginning at the up-stream end read all the channel gauges, the Francis weir-gauges, and the down-stream and up-stream spillway crest gauges in turn, making three complete circuits. The levelman recorded all readings. The observers then changed duties and repeated the series, reading the gauges first and ending with the level-rod readings. The level of the water in the submergence pool was read only once, after all other readings were completed. Frost in the connections to the stilling-wells for the spillway crest gave considerable trouble, and these gauges could not always be read. The gauge-wells for the measuring weir were outside the structure and were more accessible for clearing out ice. It was necessary each morning to drill the ice out of the connections and to thaw out the wells with hot water or by building fires around them.

Comparative Water-Surface Profiles.—The test was planned primarily as a check on a previously devised theory, and the most advantageous method of presenting the results seems to be in a series of profiles showing measured and computed water surfaces. (See Figs. 16, 17, and 18.) The high, low, and average water surfaces, determined from level-rod readings, are shown. Where available, the results of the trough-gauge readings are also platted. The down-stream part of the channel being on a level grade, the location of the control was not difficult. Where a control existed, it was found theoretically and actually to be close to the outfall end of the channel. In Runs Nos. 3, 9, 10, 12, 14, and 15, the submergence at the outfall was such that the critical velocity was not attained at any point and no control existed. In these cases, the measured water surface in the lower end of the channel was taken as a starting point.

Apparent Swell.—The computed profile generally is of the same form as the average observed profile, lying slightly below it. As noted in the theoretical discussion, it was expected that the measured depths would exceed the computed depths on account of entrained air and swell due to the discrepancy between mean and average velocities. Many factors may influence the amount of swell. The volume of air taken into the stream probably increases with the quantity and fall of the inflow. The proportion of this air held in suspension probably depends on the volume and velocity of the channel stream. The velocity swell is probably influenced by the same conditions. The data available are neither sufficiently accurate nor sufficiently extensive for developing a law for the determination of the increase in volume due to these causes. The apparent swell for the fifteen runs made at Bellvue is shown graphically in Fig. 19. The points on the graphs were determined by comparing the channel area for the computed depth with that for the average depth from rod readings. Small uncertainties in the rod readings render the results erratic, but there is a general tendency to substantiate the expected results. For example, the weir crest is unsubmerged in Runs Nos. 1 and 8, and the channel outfall is free, allowing high velocities and small storage volume. As might be expected,

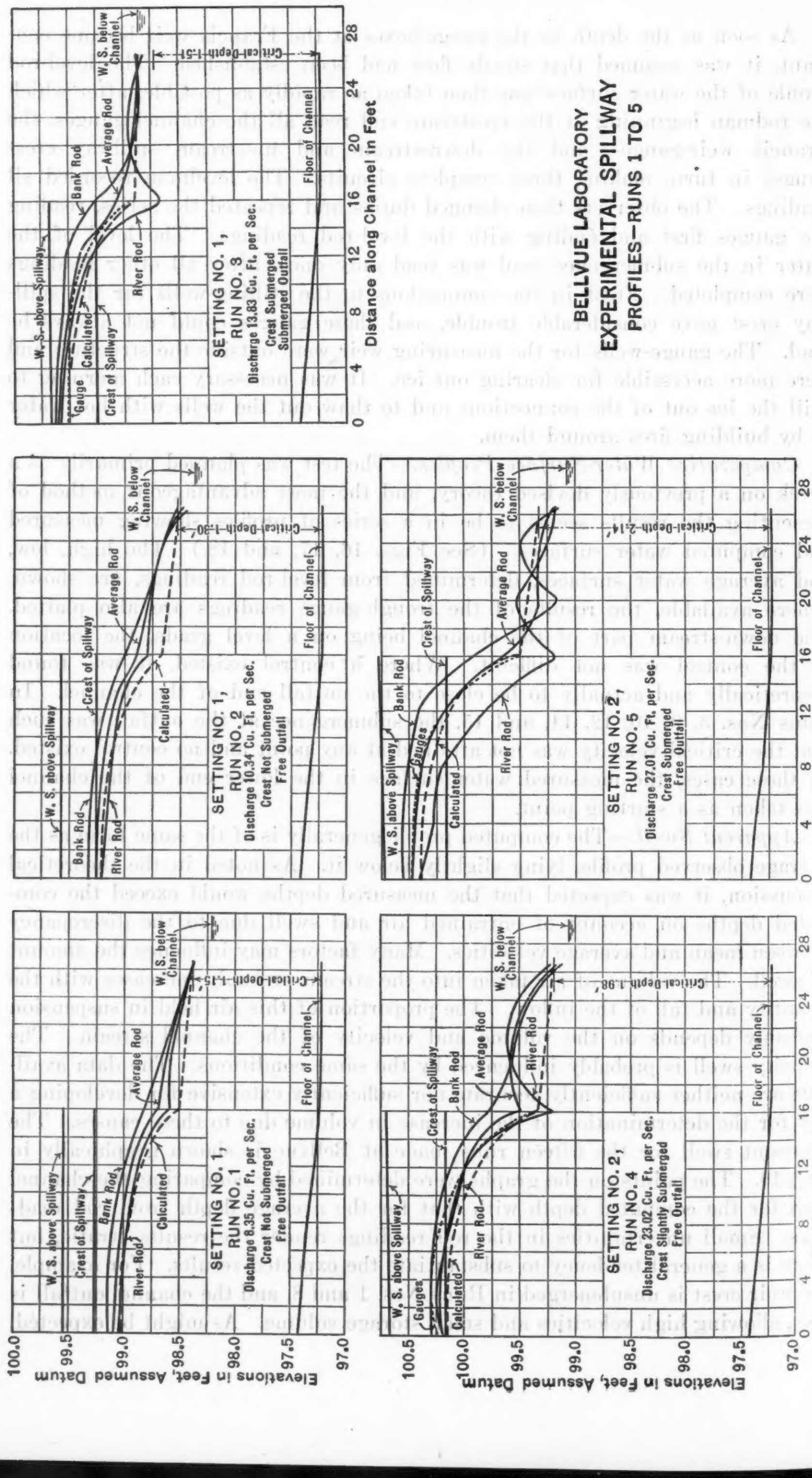


FIG. 16.—EXPERIMENTAL SPILLWAY, BELLVUE LABORATORY, PROFILES OF RUNS NOS. 1 TO 5.

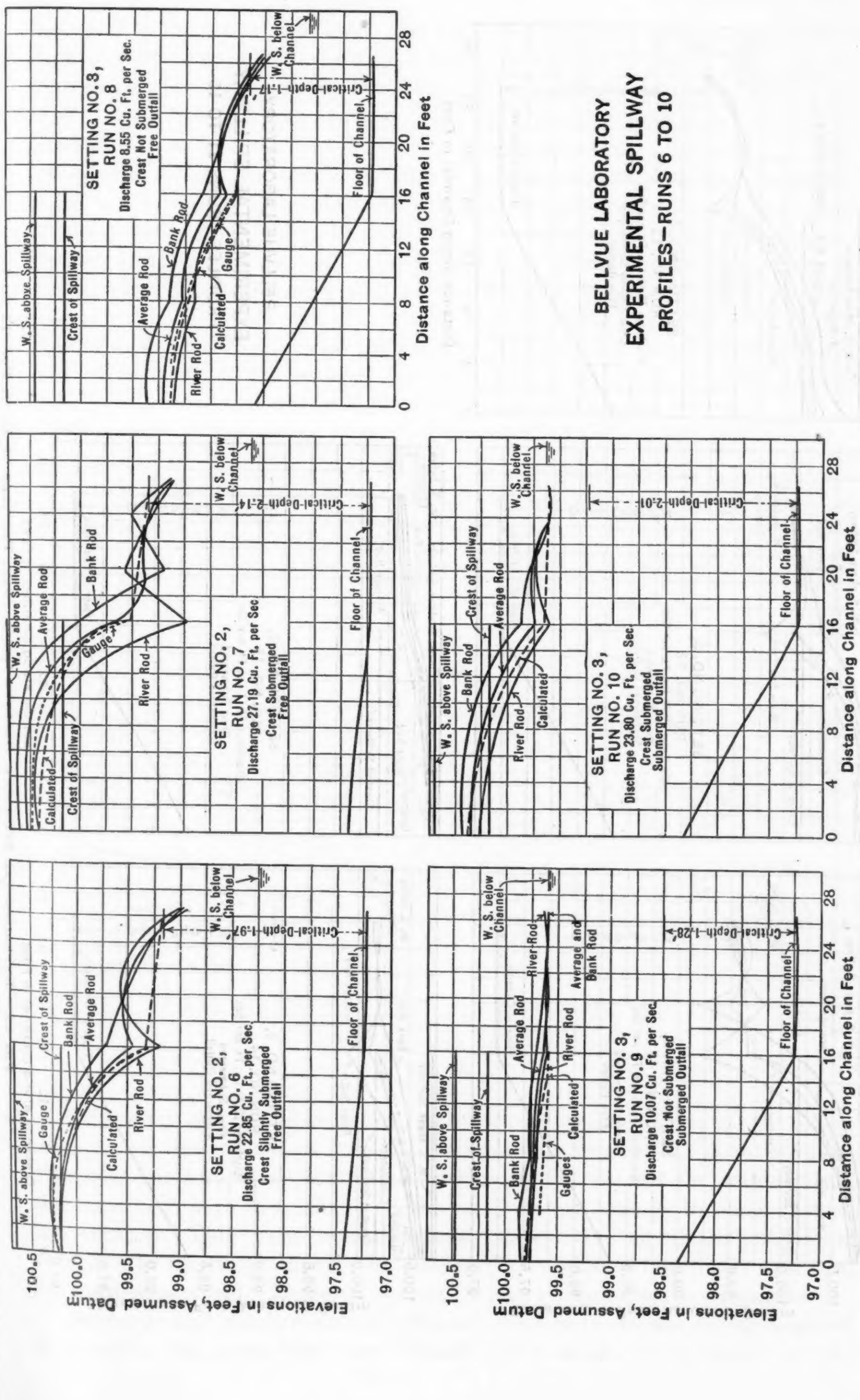


FIG. 17.—EXPERIMENTAL SPILLWAY, BELLVUE LABORATORY, PROFILES OF RUNS NOS. 6 TO 10.

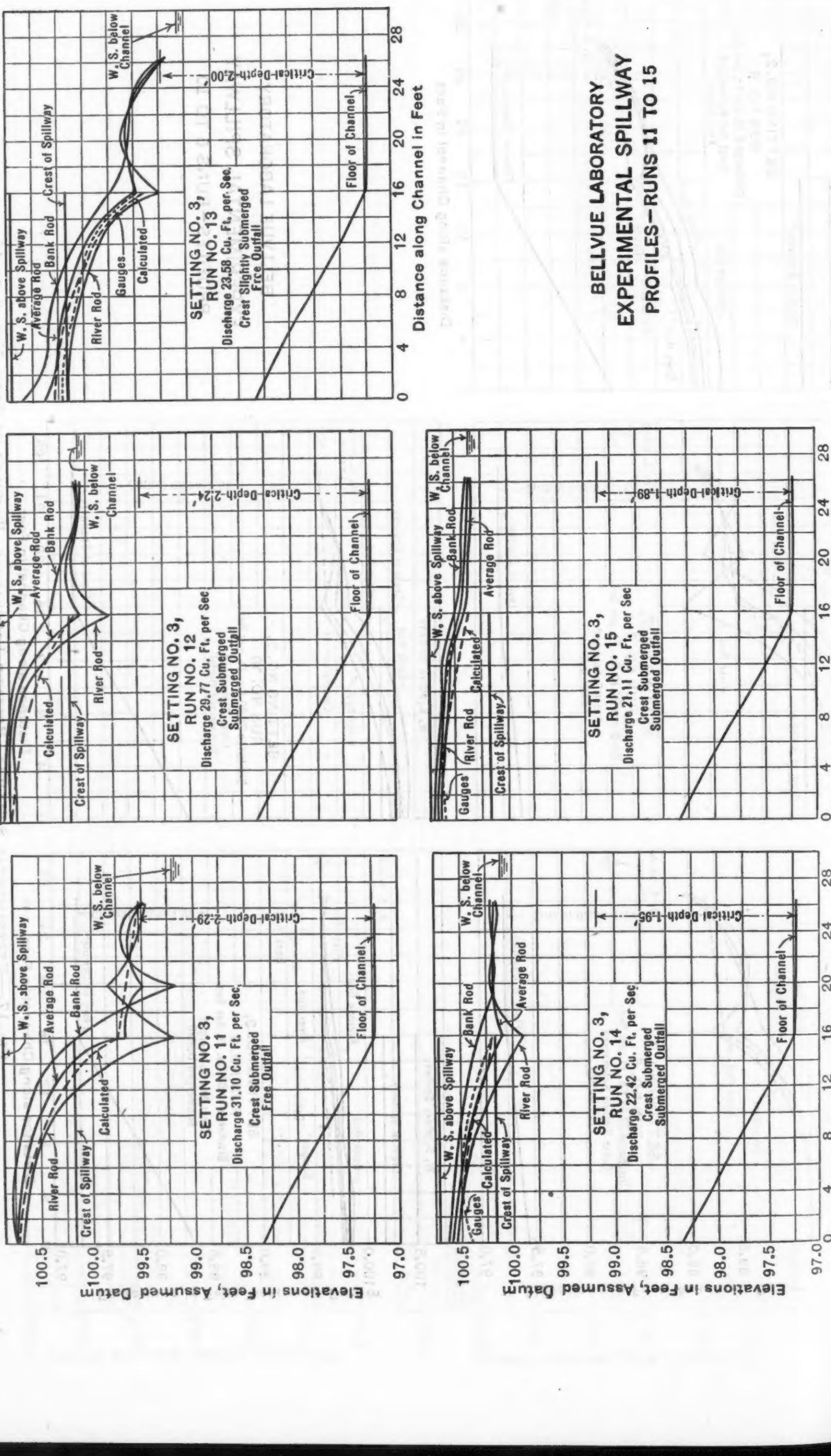


FIG. 18.—EXPERIMENTAL SPILLWAY, BELLVUE LABORATORY. PROFILES OF RUNS NOS. 11 TO 15.

the swell is comparatively high. In Runs Nos. 2, 4, 6, and 13, the crest is slightly submerged and the outfall is free. At Station 16, Runs Nos. 2 and 4 show fairly high air contents. Otherwise, these four runs show only a moderate swell. Runs Nos. 5, 7, and 11 show a heavily submerged crest with free outfall. Runs Nos. 5 and 11 show moderate and consistent swells. Run No. 7 is consistent, but higher than might be expected. The crest in Run No. 9 is unsubmerged, but the outfall is obstructed so that channel velocities are low. The swell is small. In Run No. 10, both the crest and the outfall are slightly submerged, and the swell is small. Runs Nos. 3, 12, 14, and 15 are fairly heavily submerged, both at the crest and the outfall, and, in general, show small or moderate swells. Run No. 13, which probably approximates usual design conditions as closely as any of the runs, shows an average swell of about 5 per cent. The effect of the scale of the structure on the swell is undetermined, and the test results should be applied to actual design with caution.

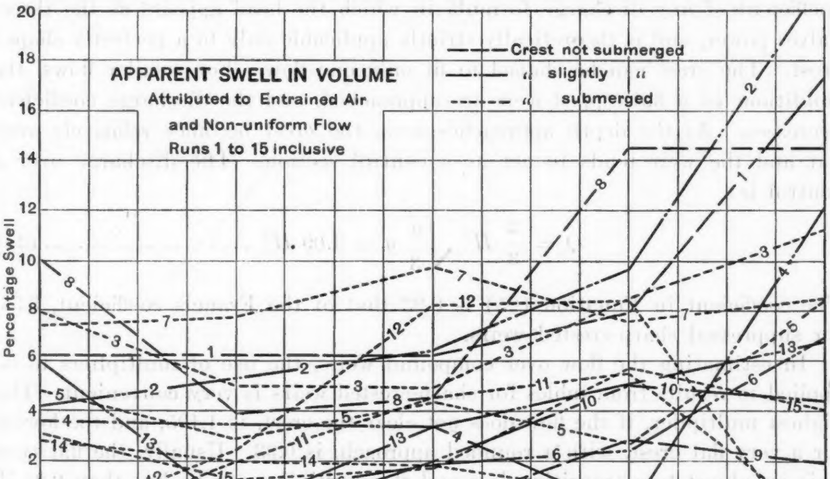


FIG. 19.

A Peculiar Air Condition.—An interesting phenomenon was discovered in studying air conditions. A "rope" of air varying in size, but about 2 in. in diameter, was formed approximately at the gravity axis of the water prism, and remained there throughout all the tests, except perhaps in cases of deep submergence. This "rope" ended squarely against the up-stream end of the channel. It increased slightly in diameter and became less distinct toward the down-stream end. If broken up with a hoe, it immediately reformed.

Discharge and Submergence Coefficients for Crest.—It was not anticipated in planning the tests that any valuable data would be obtained on the

discharge coefficient of the rounded metal crest, and no special attempt was made to obtain favorable approach conditions. Nevertheless, the results roughly check, in an interesting way, a theory of compound weir discharge previously proposed. According to Bazin's measurements* the lower nappe of the discharge over a sharp-crested weir rises above the crest elevation by an amount equal to 0.112 times the head on the crest. If a crest having the exact shape of the nappe is put under the falling sheet, it may be assumed that the discharge will not be altered. However, the summit of the new crest will be above the original crest level by the previously mentioned percentage of the head on the crest. If for convenience it is desired to express the discharge in terms of the depth on the new crest, the discharge coefficient must be multiplied by $\left(\frac{1}{1 - 0.112}\right)^{\frac{3}{2}} = 1.195$. This factor should apply to the coefficient of any discharge formula in which the head appears as the three-halves power, and is theoretically strictly applicable only to a perfectly shaped crest. The crest can be shaped to fit only one flow. For smaller flows, the conditions of a flat-topped weir are approached, and the discharge coefficient decreases. As the depth approaches zero, the crest becomes relatively very flat and the weir tends to act as a control section. The discharge over a control is:

$$Q = \frac{2}{3} H^{\frac{3}{2}} \sqrt{\frac{2}{3}\, g} = 3.09\, H^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (25)$$

The coefficient in Equation (25) is 0.93 that of the Francis coefficient, 3.33, for suppressed sharp-crested weirs.

In estimating the flow over compound weirs, the use of multipliers to be applied to results from tables for sharp-crested weirs is very convenient. The highest multiplier, if the flow does not clear the crest, is 1.195, and the lowest for a very flat crest, with a rounded approach, is 0.93. Usually, the flat type weir is subject to a transition loss and the multiplier can be less than 0.93 if the approach is not properly rounded. Multipliers for a number of crest forms have been published by the U. S. Bureau of Reclamation.† The highest value shown is 1.169. Crest multipliers for the test structure are platted in Fig. 20 (a) over depths as abscissa, no allowance being made for submergence. The figures to the left of the hyphen at each point designate the number of the run and those to the right show the percentage of submergence at Station 0 on the basis of the computed depths. It will be noted that the points for zero submergence fall quite regularly along a curve joining the theoretical points at the two extremes. The submerged points are distributed in a fairly regular way, but are insufficient in number to permit the derivation of a law governing the effect of submergence.

It was expected that the law of submerged discharge would differ materially from that for a sharp-crested weir. In order to test this assumption the theoretical discharges were recomputed, making allowance for the submergence

* See Parker's "Control of Water", p. 400.

† "Hydraulic and Excavation Tables", p. 96.

due to the computed water-surface profile in the channel, in accordance with Herschel's coefficients.* The results are shown in Fig. 20 (*b*). The figures designate the number of the runs. The points follow a definite line, indicating that the multipliers drop away from the theoretical maximum toward the control section value as the head on the crest decreases. There is no characteristic difference between submerged and unsubmerged runs. Run No. 15 is the only one that is seriously erratic. All multipliers are based on Bazin's formula for suppressed weirs, with allowance for velocity of approach, except for the lower theoretical point. Bazin's formula does not extend to a zero depth. Until more complete data are available, it appears that a multiplier of 1.15 may be safely used for a correctly shaped crest, and that Herschel's coefficients for submergence apply with sufficient accuracy to spillway conditions.

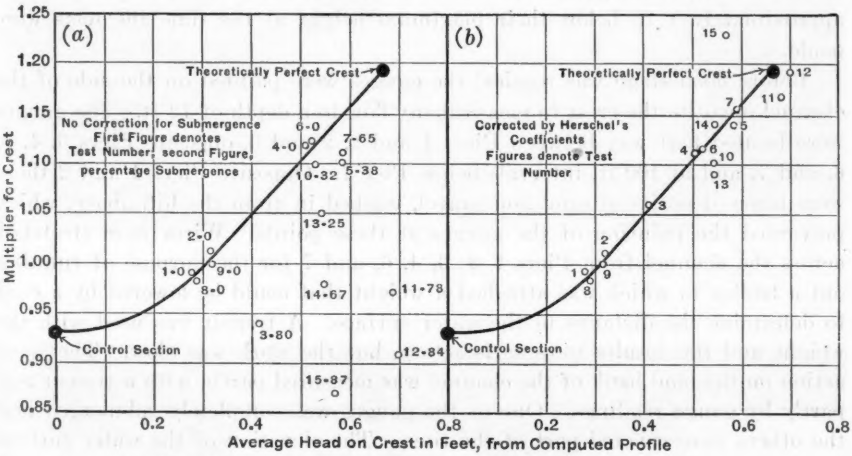


FIG. 20.—CREST MULTIPLIER AND EFFECT OF SUBMERGENCE.

Characteristic runs are shown on Figs. 21, 22, 23, and 24. The effect of submergence on the turbulence of flow is illustrated in Figs. 21 and 23, Runs 13 and 15, respectively. Conditions in these two views are approximately the same, except as to submergence at the outfall.

EXPERIMENTS AT ARROWROCK DAM

The 1923 Overflow.—A spillway structure of adequate capacity is rarely required to carry the full discharge for which it is designed. For that reason tests on full-sized installations are difficult to obtain. In the spring of 1923, the Arrowrock Reservoir, on the Boise Irrigation Project, in Idaho, overtopped the spillway by about 10 000 sec-ft. The crest of this spillway is a movable type and might have been lowered to obtain full spillway discharge. However, such a discharge is in excess of any flood ever recorded in the river, even before the construction of the reservoir, and would be considered more or less of a calamity. The designed capacity is 40 000 sec-ft., or about four times the 1923 overflow. The crest gates were not fully lowered, but the

* See Merriman's "Treatise on Hydraulics."

water spilled over them in a partly raised position. The resulting conditions were very different from those contemplated in the foregoing discussion, especially in regard to height of fall, turbulence, and air content. However, it was considered desirable to make such rough observations as were possible with simple equipment.

Preparations and Equipment.—The work was in charge of W. G. Steward, Assoc. M. Am. Soc. C. E. Measurements on two different dates were made of the water surface profile in the channel, of the draw-down to the weir, of the upper nappe of the overfalling sheet, and of the surface velocities in the spillway channel.

The general arrangement and dimensions of the spillway are shown in Fig. 25. The crest is divided by piers into six sections, each controlled by a 62-ft. by 6-ft. drum-gate. As shown on the diagram the gate crests were approximately 1 ft. below their maximum height at the time the tests were made.

Before flood stage was reached the gauges were painted on the side of the channel opposite the crest to measure any flow to a depth of 12 ft. The gauges were located half way between Piers 1 and 2, 2 and 3, opposite Piers 3, 4, 5, 6, and 7, and at 100-ft. intervals below Pier 7. Opposite Piers 1 and 2 there were heavy deposits of sand and gravel, washed in from the hill above, which prevented the painting of the gauges at these points. Wires were stretched across the channel from Piers 1, 2, 3, 4, 6, and 7 for the purpose of running out a trolley to which was attached a weight that could be lowered by a cord to determine the distance to the water surface. A transit was used with the weight and the results were satisfactory, but the work was slow. The wave action on the land bank of the channel was measured partly with a transit and partly by gauge readings. One of the gauges was completely submerged and the others were covered part of the time. The elevation of the water surface on the reservoir side of the channel was determined by a weight suspended over the parapet wall, making allowance for the inclination of suspension. The curvature of the water surface above the weir crest was measured down from the tops of the piers, and the curvature of the nappe of the falling water was measured by an extension lever with a movable weight suspended near the end.

The elevation of the movable crests was found by measuring down from the tops of the piers. The velocity of the water in the channel was determined by surface floats consisting of pieces of driftwood or bundles of kindling. The velocity of approach to the weir crest was measured with a current meter.

All measurements were made by Mr. Steward, and a report giving in detail the methods used is on file in the office of the Chief Engineer of the U. S. Bureau of Reclamation at Denver, Colo.

Comparative Profiles.—The observed water surface profiles, in comparison with the computed depths, are shown on Fig. 25. The theoretical profiles shown were not calculated from Equations (23) and (24), but from a previously devised more complicated formula based on the same theory. On May 26, 1923, the water surfaces in the channel were measured along the land bank and

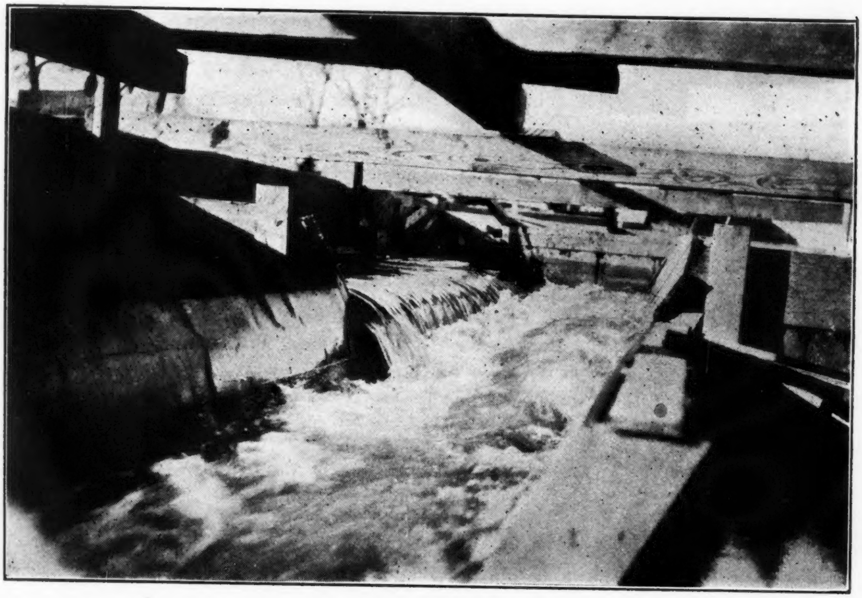


FIG. 21.—RUN NO. 13. DISCHARGE, 23.58 SEC-FT. PRACTICALLY NO SUBMERGENCE.

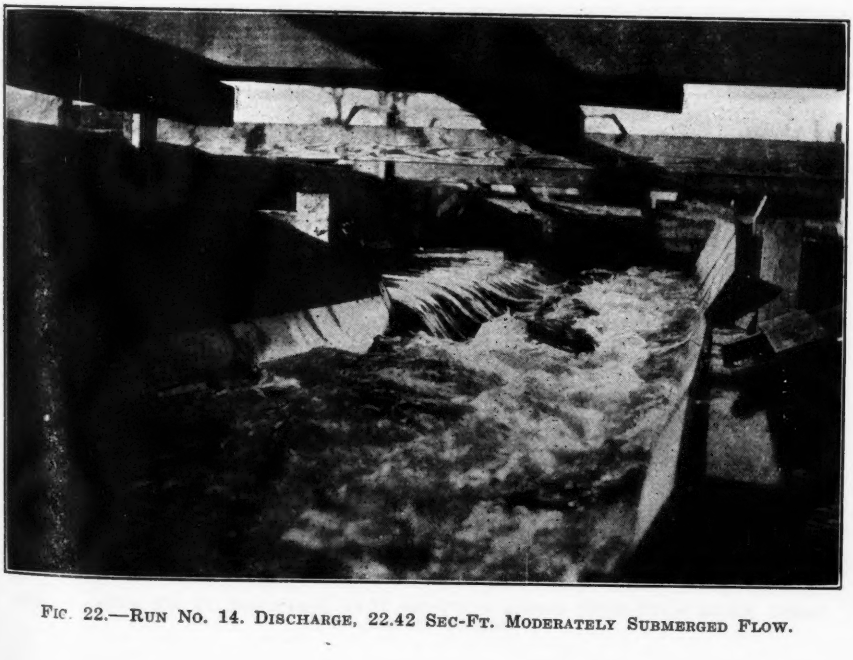


FIG. 22.—RUN NO. 14. DISCHARGE, 22.42 SEC-FT. MODERATELY SUBMERGED FLOW.

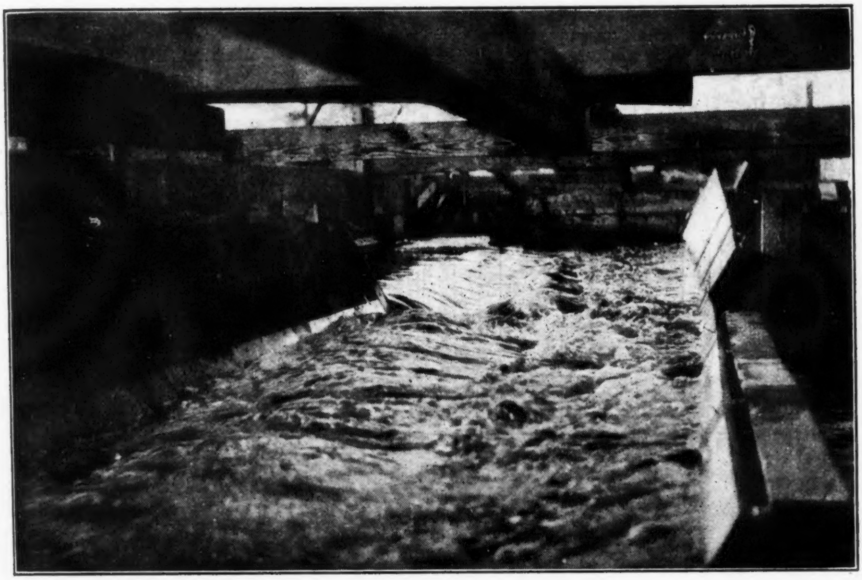


FIG. 23.—RUN NO. 15. DISCHARGE, 21.11 SEC-FT. HEAVILY SUBMERGED.

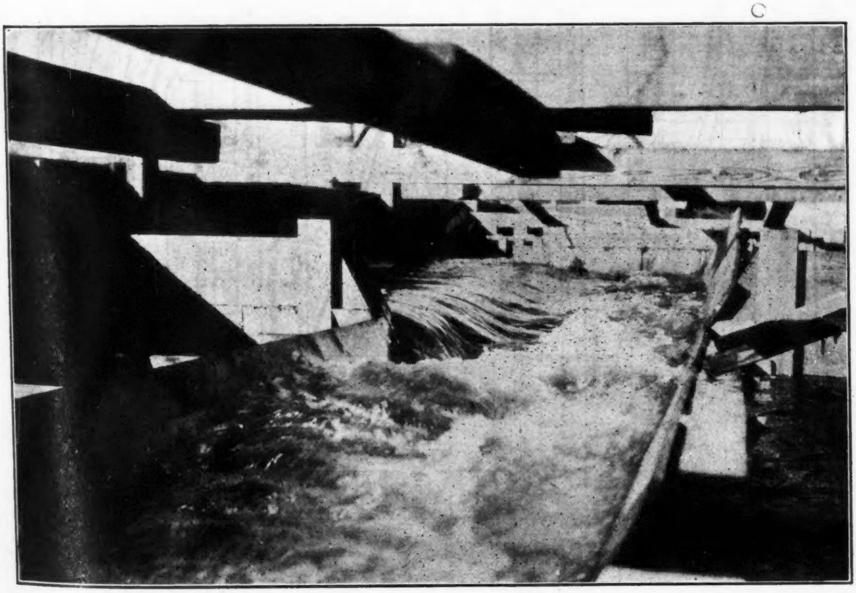


FIG. 24.—RUN NO. 12. DISCHARGE, 29.77 SEC-FT., SHOWING CROSS-FLOW BELOW CREST FOR CERTAIN STAGES.

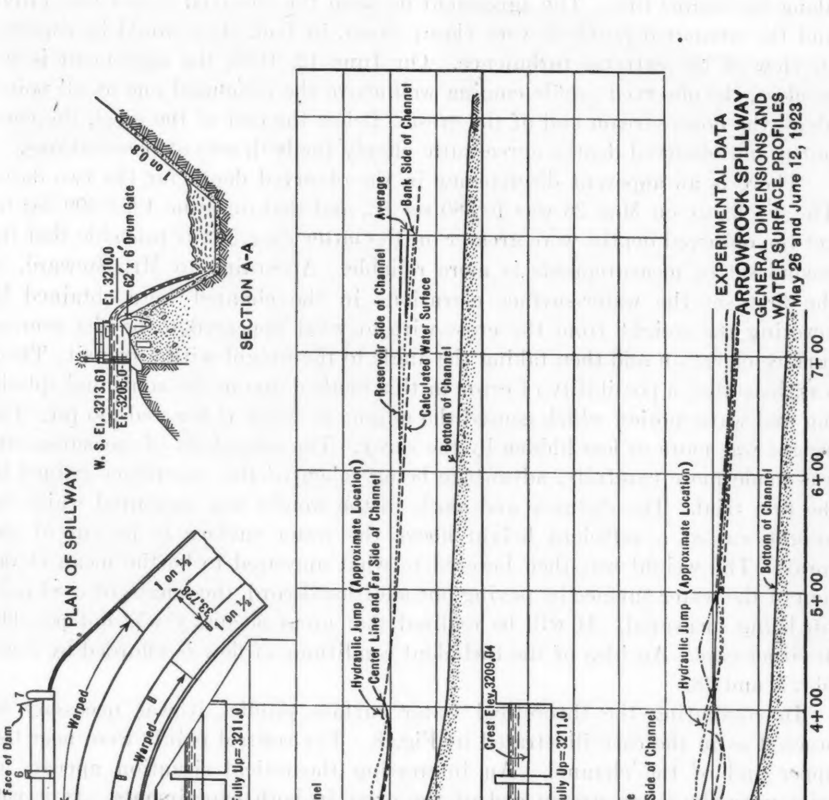

PLAN OF SPILLWAY
SECTION A-A
W. S. El. 3213.61
El. 3205.0
El. 3210.0
62' × 6' Drum Gate
1 on 0.8
Face of Dam
Pier No. 1
Gauges
Warped
DISCHARGE 10 380 CU. FT. PER SEC.
May 26, 1923
DISCHARGE 9 500 CU. FT. PER SEC.
June 12, 1923
Average Crest Elev. Gates = 3210
Crest Elev. Gates Fully Up = 3211.0
Average Crest Elev. Gates 1 to 5 = 3210
Crest Elev. 3209.04
Calculated Water Surface
Bank Side of Channel
Center Line of Channel
Reservoir Side of Channel
Average
Hydraulic Jump (Approximate Location)
Center Line and Far Side of Channel
Bottom of Channel
Elevation in Feet
EXPERIMENTAL DATA
ARROWROCK SPILLWAY
GENERAL DIMENSIONS AND
WATER SURFACE PROFILES
May 26 and June 12, 1923

FIG. 25.

along the center line. The agreement between the observed center-line profile and the computed profile is very close; closer, in fact, than would be expected in view of the extreme turbulence. On June 12, 1923, the agreement is not so close, the observed profile coming well above the computed one at all points above the down-stream end of the crest. Below the end of the crest, the computed and observed depths agree quite closely for both sets of observations.

There is an apparent discrepancy in the observed depth for the two dates. The discharge on May 26 was 10 380 sec-ft., and that on June 12, 9 500 sec-ft., yet the observed depths were greater on the latter date. It is probable that the second set of measurements is more reliable. According to Mr. Steward, in the first set the water-surface elevations in the channel were obtained by lowering the weight from the cross-wire to what appeared to be the average surface elevation and then taking the angle to the weight with a transit. There was, he states, a possibility of error in this method due to the continual splashing and wave motion which caused the weight to swing if lowered too far. The weight was more or less hidden by the spray. The second set of measurements was made more carefully, advantage being taken of the experience gained in the first trial. The distance and angle to the weight was measured while the weight was at a sufficient height above the water surface to be out of the spray. The weight was then lowered to what appeared to be the mean elevation of the water surface by paying out additional cord, the length of cord paid out being measured. It will be realized that great accuracy was not possible in either case. An idea of the turbulent conditions of flow is afforded by Figs. 26, 27, and 28.

In computing the theoretical water surface profiles it was necessary to proceed as in the case illustrated in Fig. 8. The control points were near the upper end of the channel. An interesting theoretical situation appears to exist near the down-stream end of the crest in both experiments. It is not possible to follow the flow through to the end of the inflow section without a hydraulic jump. The theoretical jump apparently lies within the observed water prism. There are many complicating influences at this part of the channel and any attempt to make accurate computations appears useless. Among other things the horizontal curvature of the spillway channel gives the inflow a component velocity in the direction of the channel, and, furthermore, a complete admixture of the last incoming water with channel flow probably is not accomplished for some distance. The approximate locations of the theoretical jumps are shown on Fig. 25.

Apparent Swell.—At the time the tests were planned it was thought that by observing velocities of flow in the channel an estimate of the volume of entrained air could be obtained. The only available method of measuring these velocities was by means of surface floats. The velocity being variable, it was necessary to time the floats at numerous points along the channel. The number of observers was not sufficient to permit the timing of a single float at all required points. The time of passage from the upper end of the channel to each of the various stations was determined separately by different floats, and the velocities for the various reaches were found by taking differences of these time intervals. No great dependence can be placed on the results. Even

FIG. 26.—ARROWROCK SPILLWAY, JUNE 12, 1923, LOOKING DOWN INTO CHANNEL FROM PIER NO. 1.

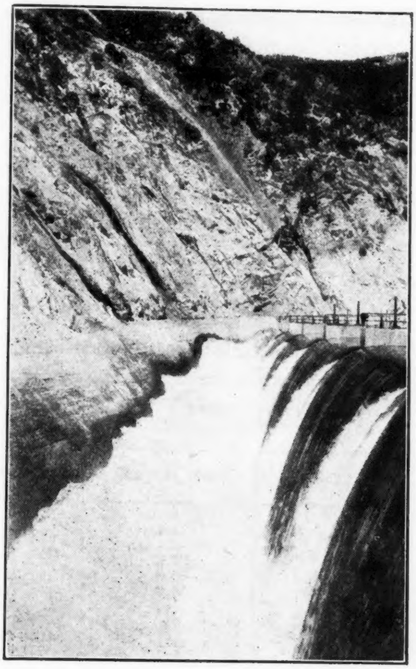


FIG. 27.—ARROWROCK SPILLWAY, JUNE 12, 1923, LOOKING UP CHANNEL FROM DOWN-STREAM END.

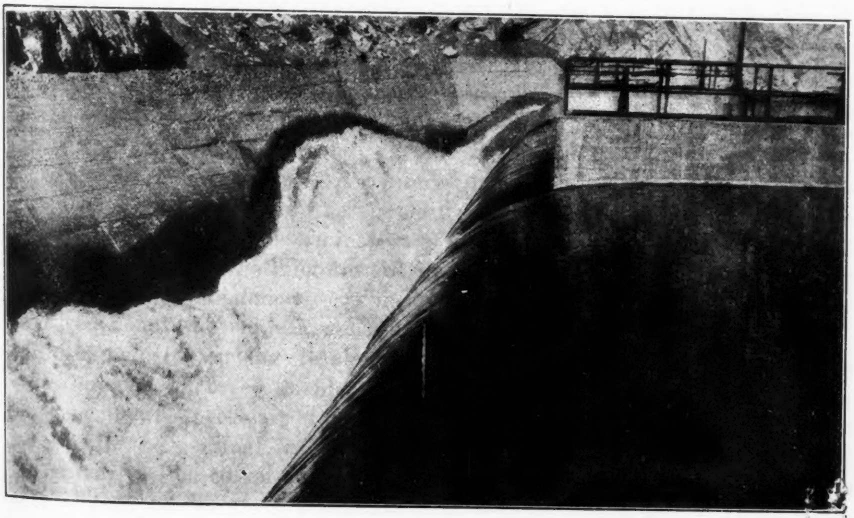


FIG. 28.—ARROWROCK SPILLWAY, JUNE 12, 1923, LOOKING UP STREAM FROM PIER NO. 4.

if the surface velocity along a given line were definitely known, it would give very little idea of the actual average velocity. A comparison of the mean velocity computed from observed depths with the surface velocity from floats gave apparent air contents of from —4% to +130% of the volume of water. The variations were erratic.

The results are shown in Table 4, as is also the swell obtained from a comparison of the observed and computed water prism areas. It is believed that the percentages of swell shown are higher than need be provided for in design.

TABLE 4.—AIR CONTENT OR SWELL, ARROWROCK SPILLWAY.

(Ratio of Air, or Swell, to Theoretical Water Volume.)

Station* numbers	From Observed Surface Velocities		From Computed Depths	
	May 26, 1923	June 12, 1923	May 26, 1923	June 12, 1923
1	....	....	....	....
2	−0.04	0.56	....	0.45
3	0.21	0.46	0.02	0.57
4	0.34	0.44	0.09	0.54
5	0.27	0.38	0.05	0.43
6	0.19	0.46	....	0.33
7	0.08	0.50	....	....
8	0.09	0.07	....	....
9	0.04	0.19	....	....
10	0.04	0.32	....	....
11	0.01	0.29	....	....

* Numbers of observation stations, not distances. (See Fig. 29.) Values show very rough approximations.

Approach Velocity to Crest.—The observed surface velocities and the approach velocities to the weir crest are shown on Fig. 29. The approach velocity measurements were taken on June 12, 1923, from Pier 5. The velocity of approach is affected by the contour of the reservoir floor near the crest and by the position of a log boom up stream from the crest. It is not the same for all stations along the crest.

Shape of Upper Nappe.—An interesting result of these observations is shown on Fig. 30. One of the simplest problems in experimental hydraulics would appear to be the determination of the path of a jet over a sharp-crested weir, yet practically nothing toward this end has been done. Measurements out to a limited distance were made by Bazin, and various writers have theorized about the subject, generally using these measurements as a basis, with the widely varying results shown in the diagram. The trajectories shown are not all based on identical assumptions, and in order to plat some of them it was necessary to make certain interpretations which may not agree with the original intention of the various authors. The majority of the curves are based on an assumed sharp crest with a vertical up-stream face. These curves are presented, not as a criticism of any of the theories proposed, but to show the lack of definite information on the subject. Mr. Steward was fortunate enough to obtain measurements of the upper nappe for

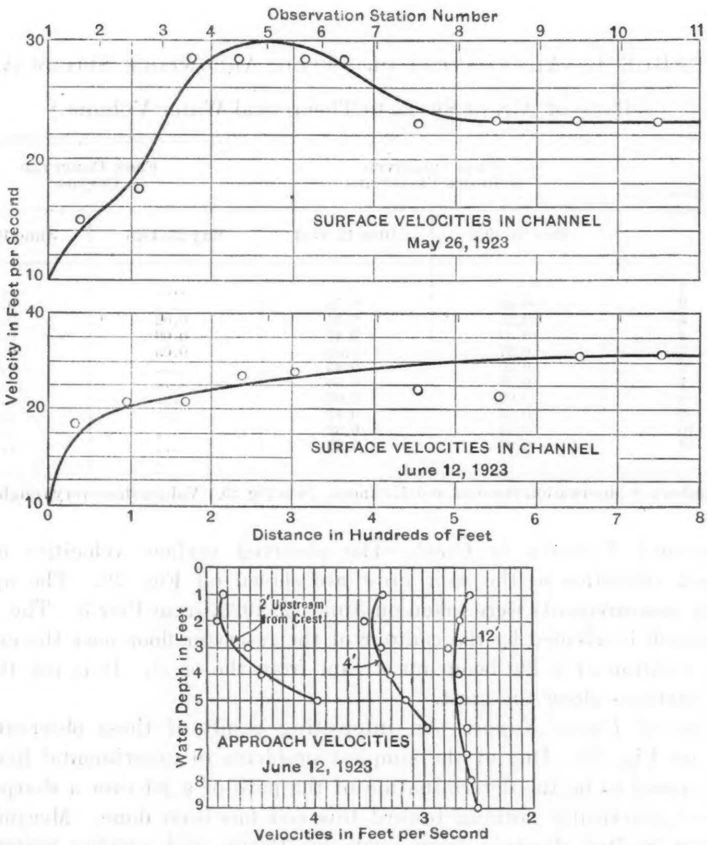


FIG. 29.—EXPERIMENTAL DATA, ARROWROCK SPILLWAY. SURFACE AND APPROACH VELOCITIES.

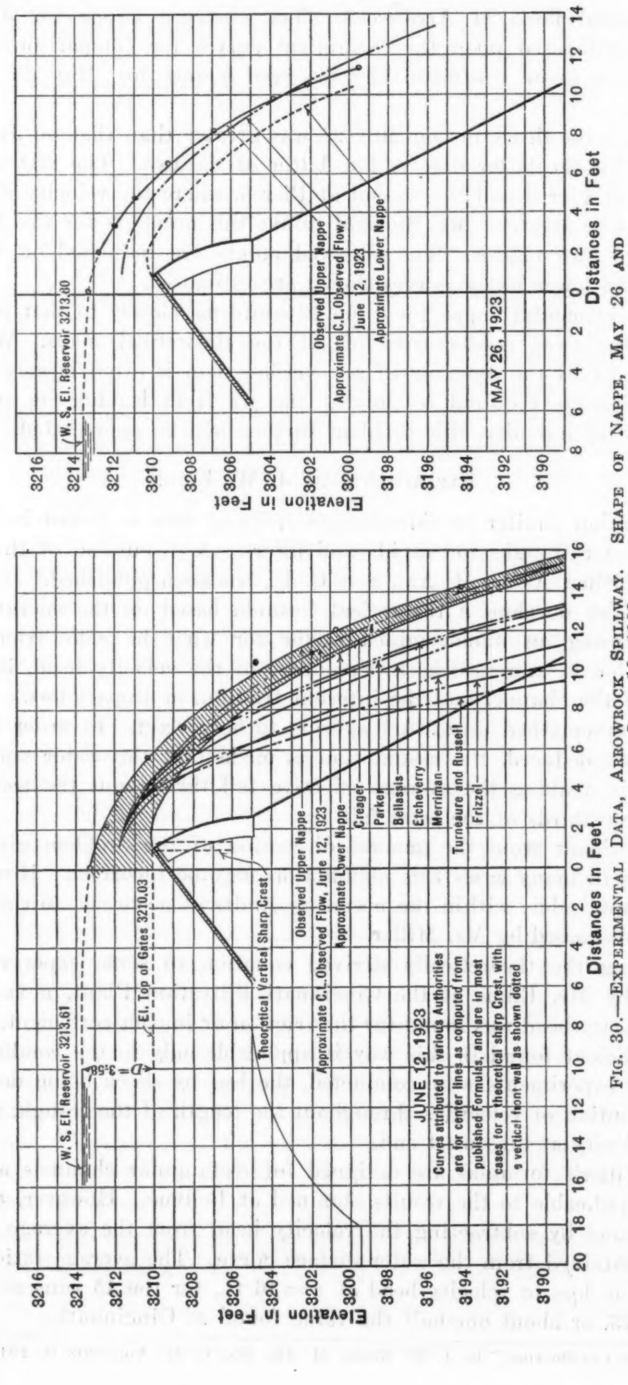


FIG. 30.—EXPERIMENTAL DATA, ARROWROCK SPILLWAY. SHAPE OF NAPPE, MAY 26 AND JUNE 12, 1923.

both measured flows at Arrowrock. The observed nappe for June 12 is shown superimposed upon the theoretical curves for comparison. In order to avoid too great confusion the observed nappe for May 26 is platted separately.

Although the discharge on May 26 was greater than that on June 12, the trajectory for the latter date is the flatter of the two. The first observation was made at Pier 3 and the second at Pier 5, where the velocity of approach appeared to be greater. Mr. Steward thinks this accounts for the discrepancy between the two nappes. The observed points are indicated on Fig. 30 by circles. The lower nappe curves shown are estimated.

The experimental nappe for June 12 conforms closely to that proposed by Creager, the most conservative of all the theoretical maps. When it is remembered that the economy of an overflow dam is often largely influenced by the thickness required to prevent the jet from leaving its surface, the importance of pursuing this problem further will be appreciated.

Experiments by J. W. Ellms

A situation similar to side-channel spillway flow is found in the design of wash-water troughs for rapid sand filters. A discussion of this problem, by C. N. Miller, Assoc. M. Am. Soc. C. E., has been published.*

Mr. Miller develops a theoretical formula based on the energy equation, and containing an undetermined factor for what he calls friction. This factor, which is expressed as a percentage of the velocity head, includes the friction of the channel and the internal friction, or impact loss.

The mathematical discussion appears to be correct. In order to simplify the formula deduced, the assumption is made that the water surface curve is parabolic, making the average effective fall throughout the trough length equal to two-thirds of the total fall.

It is evident from the general discussion of the problem given in this paper that in many cases this assumption may be incorrect. However, it is probably applicable, within the necessary limits of accuracy, to the particular problems discussed by Mr. Miller.

Applying the theoretically derived equation to some experimental data obtained by Mr. Ellms at the Cincinnati Filtration Plant, a value of 0.75 of the velocity head was found for the friction or impact coefficient.

A coefficient found in this way is applicable only to the conditions under which the experiments were conducted, the loss by shock being dependent on the distribution of velocities throughout the length of the trough, rather than on the velocity at the outlet end.

Mr. Miller's formulas are designed for rectangular channels and are not directly applicable to the results obtained at Bellvue. However, the net loss can be found by subtracting the velocity head from the average fall, which latter is obtained from the water-surface curve. The average ratio of impact and friction loss to velocity head at $x = 16$ ft., for the 15 runs at Bellvue, is about 0.375, or about one-half the value found at Cincinnati.

* "Water Purification," by J. W. Ellms, M. Am. Soc. C. E., Appendix B, 1917 Edition.

PERMISSIBLE CANAL VELOCITIES

By Samuel Fortier* and Fred C. Scobey,† Members, Am. Soc. C. E.

Synopsis

The use of relative high velocities reduces the cost of canal per unit length. The limit of velocities permissible is somewhat less than velocities that will erode a canal bed. The determination of permissible velocities is not possible from data on transporting velocities or mere non-silting velocities. The presence or absence of colloidal matter is of prime importance. The opinions of irrigation engineers as determined from their experience are given, as well as deductions from available data and the final recommendations of the Special Committee on Irrigation Hydraulics.

Definitions.—Limiting velocity is taken to mean the maximum permissible value of the mean velocity. Although it is obvious that for any water prism the highest velocities are much in excess of the mean velocity, yet the only figure sufficiently tangible under all conditions, for design and operation, is the mean velocity.

Silt is not considered as a specific gradation in soil sizes, but as a general term for all material brought into a canal by "muddy" waters. It is usually thought of, even in this sense, as different from sands and gravels. In other words, the "bottom load" is not included in the term as herein used.

Velocity and Canal Length.—As the discharge is the product of area and mean velocity, it follows that the area of the water prism, and, therefore, the approximate area of the channel, must vary inversely with the mean velocity. As the cost of construction per unit length depends on the area of the channel, it is usually desirable to make the mean velocity as high as the material will withstand. The cost of construction also varies with the length of the channel. By increasing the grade of the channel the velocity of water therein is increased and the water prism diminished, but the distance between the irrigable land and the diversion point is increased. If the point of diversion is fixed, a higher diversion dam will be required in order to command the same area of land. This phase of the question requires a comparative study for each location, whereas the purpose of this paper is to indicate maximum feasible mean velocities.

Note.—This paper constitutes the Final Report of the Special Committee on Irrigation Hydraulics on this subject, reference to which is made in the Progress Report of that Committee for 1924, *Proceedings*, Am. Soc. C. E., March, 1925, Society Affairs, p. 137. Discussion on the subject is invited.

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† Senior Irrig. Engr., U. S. Dept. of Agriculture, Berkeley, Calif.

Non-Silting Velocities Not Necessarily Scouring Velocities.—The subject of limiting velocities for open channels has usually been treated under a heading combining the silting and scouring of channels. It has been considered that for each depth of water there is a certain velocity below which silt will be deposited and above which silt will be eroded from the bed of the channel. In the opinion of the writers, two phenomena only slightly related are thus confused. The power of flowing water to maintain a movement of separate soil, sand, or gravel particles, has been confused with the power to break the bedding of a canal bottom and ravel off particles of that bed and transport them to places of lesser velocities where they may be deposited. In their opinion there is no sharp line of demarcation between the velocities that can no longer maintain silt in movement and those that will scour a canal bed. It is believed that there is a broad belt of velocities between these two "critical" velocities, within which silt already loosened or brought in through a head-gate will remain in suspension while the bed nevertheless will remain undisturbed as regards scour. It is easy to show the absurdity of accepting the laws of silting as giving the immediate answer to the laws of scour.

Kennedy's Formula.—Certain of the canals of India, served by heavily silt-laden streams, were known to have reached a condition of permanent channel régime, neither silting nor scouring. In 1895, Mr. R. G. Kennedy offered his well-known formula*:

$$V_0 = C\, d^{0.64}$$

in which, V_0 represents the "critical" mean velocity, in feet per second, of a water prism, d ft. in depth, and C is a coefficient. This so-called critical velocity is that for which there was no silting or scouring of the channel. The value of C is usually quoted as 0.84, but this coefficient was applicable for fine sand silt only; other coefficients for heavier materials range up to 1.09 for coarse silt or detritus from hard soils. Using the coefficient, 0.84, Table 1 may be developed.

TABLE 1.

Depths, in feet.	1.	2.	3.	4.	5.	6.	7.	8.	9.
Velocity, in feet per second.	0.84	1.30	1.70	2.04	2.35	2.64	2.92	3.18	3.43

A glance at Table 1 shows the fallacy of accepting these velocities as the lower limit of scouring velocities. Mean velocities of 2½ to 3 ft. per sec. for general conditions are considered as most desirable in irrigation canals of the United States, yet Table 1 would indicate that a canal, 6 ft. deep, is the shallowest which would not show scour at the desirable velocities mentioned in the table. Yet even the velocities as tabulated are further modified, according to Bellasis,† who quotes rules attributed to Kennedy to the effect that:

* *Minutes of Proceedings*, Inst. C. E., Vol. CXIX (1895).

† "Irrigation Works", 1913, New York and Lond., p. 48.

"Near the hills [Northern India] where the bed is of shingle the velocity may exceed V_0. A few other soils will stand 1.1 V_0. In ordinary channels any excess over V_0 will give much trouble lower down. * * * If soil is very poor, especially if the depth of water is more than 6 or 7 ft., the velocity should be less than V_0, say, 0.9 V_0, so as not to cause falling in of the banks * * *."

A comparison between the figures in this quotation and the values of V_0 in Table 1 shows that the velocities resulting from such procedure would not be acceptable to engineers designing and operating canals in the United States. They would be considered much too low.

Dubuat and Gilbert on Transporting Velocities.—Reference to the earlier writings of accepted American authorities shows that they applied the findings of Dubuat to the determination of the permissible velocities for ditches and canals. These velocities are usually less than one-half the velocities now accepted by irrigation engineers. Dubuat made certain determinations of the transporting power of water. In the words of Gilbert "what he really investigated was chiefly competence for flume traction." The "competent bed velocity," as ascertained by Dubuat for various materials is given by Gilbert* as shown in Table 2.

TABLE 2.

	Feet per second.
Potter's clay	0.27 to 0.35
Coarse angular sand	0.7 " 1.1
River gravel:	
Size of anise seed	0.35 " 0.53
Size of peas	0.62 " 0.7
Size of common beans	1.1 " 1.55
Rounded pebbles, 1 in. in diameter	2.1 " 3.2
Angular flints, size of hen's eggs	3 2 " 4.0

Although the velocities along the bed of a flume, such as that used by Dubuat, would be less than the mean velocities, still the difference would not be sufficient to account for the great difference between the figures of Table 2 and those given by various authorities as determined from actual practice.

Material Easily Transported Is Difficult to Scour.—Additional evidence against confusing the movement of particles by flowing water with the scouring action of that same water is contained in the following: The finer the soil particles, the slower may be the velocities that will maintain the particles in suspension. In actual practice it is found, as noted in the suggestions given hereafter, that canal beds in stiff clays, classed by soil experts as being composed of particles of 0.005 mm., or less, in diameter, will withstand mean velocities approximating 4 ft. per sec., yet clear sand coarser than granulated sugar will be scoured at velocities of about one-half that, or 2 ft. per sec.

Colloidal Matter.—Some alluvial silts or loams will erode quite easily, when a canal is new, whereas some, of apparently the same size gradation,

* "The Transportation of Débris by Running Water", by G. K. Gilbert, *Professional Paper No. 86*, U. S. Geological Survey, Wash., 1914, p. 216.

will not. The difference lies mostly in the presence or absence of effective colloidal matter. Colloids, as applied to soils, give the properties of plasticity, cohesion, toughness when wet, and hardness when dry, that are essential to an erosion-resisting soil. This point is so important that the writers quote extensively from that well-known authority on Western soils, Hilgard.*

"*'Colloidal' Clay.*—In connection with soils, clay may be defined, in the most general terms, as being the substance which imparts plasticity and adhesiveness to soils when wetted and kneaded, and which, when heated to redness, loses this property completely and permanently, becoming hard and coherent in proportion to the degree of heat to which it is exposed.

"In common life, however, the name is applied to the whole of any naturally occurring earth which on wetting and kneading assumes a reasonable degree of plasticity and adhesiveness. When the latter property becomes nearly or quite insensible, the earth is designated as a 'loam', more or less 'clayey' according to the amount of the pure, plastic and adhesive material associated with the mineral powders and sand that form the bulk of most soils. * * *

"It has of late been attempted to extend the meaning of this word (plasticity) to the behavior of all powders when wetted with water. But the adhesive plasticity of clay stands almost alone, in that (aside from contraction) it preserves in drying the form into which it may have been molded while wet, even when struck, whereas other powdery substances similarly treated at once collapse back into the original powder. The exclusive use of clay in modeling offers the typical example of plasticity as generally understood. The addition of any powdery substance, however fine, diminishes the plasticity of clay. * * *

"*Causes of Plasticity.*—In any case the property of plasticity and adhesiveness is restricted to the particles so fine that they fail to settle, in the course of 24 hours, through a column of pure water 8 in. high, while some are so extremely minute that they will not settle for many months, and even for several years. Such turbid 'clay water' may sometimes be found existing in nature, in moist, secluded places, for weeks after the subsidence of the overflows of rivers whose water is exceptionally free from dissolved mineral matter. * * *

"When separated from the water and dried, the jelly-like substance ('colloidal clay') shrinks as extravagantly as would so much boiled starch, into hard, shiny crusts or flakes, which when struck in mass are sometimes even resonant, and bear more resemblance to glue than to the clay of everyday life. Like glue, too, but much more quickly and tenaciously, the dried colloidal clay adheres to the tongue, so as to render the separation painful; when wetted it quickly bulges with great energy, and in a short time resumes its former jelly-like condition. When moistened with less water it assumes a highly plastic and adhesive condition, so that it is difficult to handle and almost as sure to soil the operator's hands as so much pitch."

With this understanding of soil colloids, it is easily seen that great changes may take place in a canal bed after a period of use under the influences of various types of waters. This explains the thought expressed in some of the answers to questionnaires, that old, well-seasoned canals will stand much higher velocities than new ones.

Effective Colloids.—The presence in the water of an electrolyte, such as calcium or carbon dioxide, will flocculate the colloids and partly or completely

* "Soils," by E. W. Hilgard, N. Y., 1906, pp. 59 *et seq.*

annul the property of creating cohesion. Therefore, effective colloids are those free from the influence of an electrolyte.

Types of Bedding.—The property of a canal bottom to withstand scour is due to beds of different types, of which the following are examples:

1.—Very fine silty or clayey particles so puddled together that there is distinct cohesion present; a homogeneous mass—the more colloidal matter present, the greater will be the cohesion.

2.—Graded materials, the finer filling the voids of the coarser; ranging from the finest of silts to coarse gravel. Such a bed becomes very firmly compacted and will withstand velocities that would erode a bedding composed of any of the constituent particles alone.

3.—Non-cohesive particles, without grading as to sizes. The unit particles may be of "sugar" sand or of cobble-stone size; if free from colloidal silt or graded sizes a canal bed in such materials may come nearest to following what Gilbert terms the laws of "competent velocities."

Original Channel Material Usually Altered by Age.—A study of these types shows that the particular one to which any given canal will belong may or may not be determined at the time of construction. The type may not be determined by the materials excavated for the canal. It is finally determined only after years of use. Two examples will make this clear. The first is indicative of changes caused by the character of the water diverted from the river. The second example will develop under clear mountain waters carrying little or no silt, and is due to the character of the material through which the canals are excavated.

In the Imperial Valley of California many of the large laterals were excavated in a light sandy soil that would have scoured when the canals were new at a velocity of 2½ ft. per sec. The extremely fine silt particles brought in from the Colorado River prior to 1910 compacted and puddled this sand and plastered it with a coat of slick colloidal mud, so that it would withstand velocities in excess of 5 ft. per sec.; yet the individual particles of this slick mud were so fine that they would remain in suspension at very low velocities, except when the water was "overloaded" with silt—in times of flood on the Colorado River in June and July. All experienced canal operators know the trick of holding muddy water above one check structure after another until the mud has painted over the sides and bottom of a new canal, reducing seepage losses and making the bed of the canal less susceptible to scour.

Many of the mountain valley canals of Colorado, Utah, and other Rocky Mountain States were constructed in loamy gravel soils. The present bottoms of these canals show little indication of loam. At the extreme edges, grass, roots, and low velocities combined have built up vertical banks of fine material, but the bottoms are of well-graded gravel, ranging from pea size to cobble-stones, but all so well compacted and fitted together that a high velocity is feasible, far above that which originally washed out all the loam particles until a gravel paving resulted.

Effect of Velocities on Plant Growth.—The problem of plant growth is an important one in the operation of irrigation canals and deserves special treatment. It is, however, influenced by factors other than canal velocities. More particularly these factors comprise water temperatures and turbidity and character of soil.

Few aquatic plants thrive in the absence of sunlight, therefore, canals carrying turbid waters are seldom bothered with plant growth. In clear-water channels some species thrive in velocities much greater than the channel soil will stand. Other species grow only in sluggish velocities. If, therefore, the highest permissible canal velocities as determined from the standpoint of erosion are provided, all that is possible will have been done to prevent trouble by plant growth as far as fixing velocities are concerned.

Difficulty of Securing Specific Data.—Thus far in this discussion the writers have endeavored to establish the fact that the determination of eroding velocities is a matter of empirical observation on erosion, not on silt deposition, to the time when a particle is broken off from the bed of the canal, the cohesion due to colloidal matter being overcome or the mechanical interkeying of various elements being disturbed. After that takes place, the data of Dubuat, Kennedy, Gilbert, and others may apply. The actual determination of these empirical data has never been attempted, as far as the writers are aware. That is, it has not been considered feasible to increase slowly the mean velocity in a canal of a definite material and thus determine the actual mean velocity at which scouring commenced. The only data available are the facts that certain canals have scoured under relatively high velocities. The magnitude of these high velocities is known from measurements of the canals, but the exact high velocity that started the scour has never been determined. Undoubtedly, a higher velocity is required to start scour than to continue it when once started. Likewise, it is known that certain canals do not scour even if the mean velocity is two to three times those velocities suggested by Dubuat or Kennedy. It still remains to be determined how much higher a velocity this particular canal would stand before scour commenced.

Hence, at present, the best knowledge of scouring velocities comes from personal deductions as to the performance of individual canals and not from direct experimental work. With this understanding in mind, the Committee submitted questionnaires to a number of irrigation engineers whose experience qualified them to form authoritative estimates of the maximum mean velocities allowable in canals of various materials. The questionnaire, as submitted, follows:

"QUESTIONNAIRE ON PERMISSIBLE CANAL VELOCITIES

"This list of questions is submitted by the Special Committee on Irrigation Hydraulics of the American Society of Civil Engineers in order to ascertain the results of experience as to the permissible mean canal velocities in various types of soil under normal operating conditions.

"The Committee will appreciate the results of your experience and observation on this subject.

"1. Name?

"2. To what irrigation system or systems do your answers apply?

"3. What in your opinion is the maximum mean canal velocity permissible without eroding each type of soil according to the following classification:

	Ft. per sec.	Ft. per sec.
Alluvial silts		
Fine loam		
Fine sandy loam..........		
Fine sand.................		
Volcanic ash.............		
Coarse sand..............		
Fine gravel...............		
Coarse gravel		
Stiff clays................		
Shales		

"4. If the soils with which you have had experience do not fall plainly in the above classification can you give such permissible velocities with a description of the soils to which they apply?

"5. Do you find that canal velocity has any marked effect on the growth of mosses and aquatic plants and, through them, on erosion?

"6. If you have made actual determinations of highest non-scouring velocities with mechanical analysis of soils, or if you have made an investigation of aquatic plant growth as affected by velocity of water the Committee will appreciate a copy of the results or reference to where they may be found, if published."

Opinions of Irrigation Engineers.—Constructive replies and suggestions were received from the following list of engineers, to whom the Committee acknowledges its indebtedness:

Herbert D. Newell, Assoc. M. Am. Soc. C. E., Superintendent, Klamath Project, U. S. Bureau of Reclamation, Klamath Falls, Ore.

C. N. Perry, M. Am. Soc. C. E., Consulting Engineer, Los Angeles, Calif.; formerly Chief Engineer, Imperial Irrigation District, California.

Fred D. Pyle, M. Am. Soc. C. E., Irrigation Engineer, Imperial Irrigation District; formerly Irrigation Manager, North Platte Project and Project Manager, Uncompahgre Project of U. S. Bureau of Reclamation, and General Manager, Columbia Irrigation District, Washington.

J. L. Lytel, M. Am. Soc. C. E., Superintendent, Yakima Project, U. S. Bureau of Reclamation, Yakima, Wash.

W. H. Code, M. Am. Soc. C. E., Consulting Engineer, Los Angeles, Calif.

Jerome H. Fertig, M. Am. Soc. C. E., Bitter Root Irrigation District, Montana; formerly Project Engineer, Uncompahgre Project, U. S. Bureau of Reclamation, and, later, connected with numerous irrigation districts in Oregon.

C. E. Atwood, Assoc. M. Am. Soc. C. E., Chief Engineer, Valier Carey Act Irrigation Project, Valier, Mont.

M. E. Bunger, Engineer for various companies in Colorado; formerly Hydrographer and Deputy State Engineer, Colorado.

Charles Taylor, Hydrographer, Umatilla Project, U. S. Bureau of Reclamation.
C. C. Williams, Engineer, Wenatchee Reclamation District, Washington.

From these replies Table 3 has been compiled. Much of the value of the comments of the authorities noted would be lost if Table 3 alone were given, hence quotations and excerpts are also appended.

TABLE 3.—PERMISSIBLE VELOCITIES, IN FEET PER SECOND, ON TANGENTS AS SUGGESTED BY IRRIGATION ENGINEERS IN ANSWERS TO QUESTIONNAIRES.*

Material.	C. N. Perry.	F. D. Pyle.	J. L. Lytel.	J. H. Fertig.	C. E. Atwood.	M. E. Bunger.	C. C. Williams.
Alluvial silt	2½-2¾	4.00	....	2.00	....	2.00	....
Fine loam	......	2.75	2.00	2.00	1.80	2.50	....
Fine sandy loam	......	3.00	2.00	1.50	2.50	2.50	2.00
Fine sand	......	2.50	2.00	1.50	....	2.50	2.00–
Volcanic ash	......	3.00	2.50	2.00	....	....	2.75
Coarse sand	......	2.75	3.00	2.50	....	3.00	2.00
Fine gravel	......	3.50	4.00	2.80	....	3.00	3.50
Coarse gravel	......	5.00	6.00	3-5	4.00	....	....
Stiff clays	4½-5	3.75	....	2.50	3.00	4.00	....
Shales	......	6.00	7.00	2½-4	....	3-6	....
Materials not listed in questionnaire	......	....	....	....	....	....	....
Ordinary sandy loam	......	....	....	....	....	....	2.75
Heavy clay loam	......	....	....	....	....	....	3.00

* A few of these engineers suggested velocities for curves ranging from 0.5 ft. to 1 ft. slower. It would not be practicable to change from one velocity to another as the canal went into and out of curves. It would be better to use one value for straight canals and a slightly lower one for sinuous ones.

Mr. Newell felt that the experience gained on the Klamath Project did not warrant an estimate of maximum velocities allowable, but submitted answers to questions in the questionnaire, as follows:

3. "There has been no trouble from erosion on any of the canals of the Project. Generally the velocities here used are comparatively low. The velocities listed are taken from the highest current-meter measurements of 1924.

4. "The soils encountered in canals of this Project are fine sand and a chalk that is intermediate between a stiff clay and a shale. Velocities as given below were in fine sand:

Canal.	Quantity, in second-feet.	Velocity, in feet per second.	Soil.
A	750	1.80	Fine sand
B	115	1.26	" "
C	531	1.35	" "
D	155	1.98	" "
E	18.6	1.10	" "
F	31.3	0.95	" "
C–G	249	1.03	" "
Div. Chan.	305	1.67	" "
C-4	92.3	1.62	" "

Mr. Pyle offers particularly valuable suggestions in the following:

"The velocities given are average only and are not proposed as proper to meet the various soil conditions without careful study. In general, an excess

of grade controlled by checks is preferable to a flat grade, as the condition can be remedied in the first instance and cannot be in the second.

"Some soils are very light, mushy and easily eroded when new canals are being used for the first time and will set and toughen after several years use so as to stand 50% more velocity than they would the first year. In fact, very few canals will stand velocities the first few years that they will after 10 years' use.

"Other soils such as volcanic ash and clays will erode gradually but almost constantly forming large holes in the bottom which create boils in the water. I filled a number of these while with the Columbia Irrigation District with sandy loam covered with about 3 in. of fine gravel.

"The nature of the silts carried by the water affects the erosion as some fine silts penetrate and toughen the bottom and sides of the canal, while some coarse silts tend to start erosion.

"The amount of silt carried affects the erosion as on the Imperial Irrigation District canals, where very little erosion takes place at times due to excess of silt which prevents the water from picking up more, while at other times as the water clears it will pick up silt previously deposited and commence eroding.

"In many instances a canal will run satisfactorily for a number of years without eroding, when something will occur to break the natural lining of the canal with the result that swirls and boils are formed which increase the erosion until considerable damage may result.

"Generally erosion commences on the outside of curves, around obstructions or where the current is deflected against the banks. In the last instance we are not always able to decide just why the current changes from one point to another. Back currents are very dangerous when once started and should be controlled as soon as possible."

Mr. Lytel submits a table showing the highest velocities found on some of the canals on the Yakima Project and brings out again the point that these may not be the limiting velocities. These figures are given in Table 5. He then submits an estimate of limiting velocities included in Table 3, with the following explanatory remarks:

"We find that the velocities given afford very satisfactory service, but may be increased or decreased very considerably without damage to the banks or the depositing of silt. The Yakima River is comparatively free from silt, so a great deal of attention has not been given to the silt problem in connection with the designing of canal sections.

"The figures shown in Table 5 give the velocities at gauging stations on a number of the canals along the Yakima River to which we deliver water in bulk.

"All of the stations shown in Table 5 are on tangents. The velocities given are those which are found at our regular gauging stations on the canals and at the points given. Greater velocities in several cases would probably be possible without injury to the canal section. Lesser velocities in most of the cases would be possible without silting. We have no information as to the velocities on curves, since all our gauging stations are located on tangents. In any study of this kind it is practically certain that special gaugings are necessary. Our hydrographic work has been necessarily confined to the stream measurement work necessary for the establishment and maintenance of the various gauging stations.

"On account of the erosion of the canal banks in the main Sunnyside Canal, we have some silt in the water in the lower part of the system, but not in sufficient quantity to give trouble in handling the water or increase the expense of maintenance to any extent.

"On account of the lack of silt in the water of the Yakima River, this valley is not a very good place to secure information on velocities in canals built in different material with reference to the carrying or depositing of silt, but I thought such information as our records contain might be of some use."

Mr. Code is quoted, as follows:

"The writer's experience in operating canals from the Salt River for a period of 10 years prior to the building of the Roosevelt and Granite Reef Dams taught him that with mean velocities materially under 2.5 ft. per sec. there would be deposits of silt on both sides and bottom of main canals and laterals, and all canals and main laterals built under his direction during this period were designed as far as practicable with hydraulic functions insuring approximately such maximum velocities during flood seasons. At other times there was little silt carried in the water and when the low water period occurred, our canals would be running small heads of clear water."

Mr. Atwood makes plain that the velocities given opposite his name in Table 3 are the highest found and may not be the highest attainable:

"The velocities given have been taken from our measuring stations where no erosion occurs and where the canals have given very satisfactory results from an operation and maintenance standpoint. We have not made a study of the maximum permissible velocities."

Pertinent remarks from the answers of Mr. Bunger are:

"If the water running through fine loam contains material in suspension that tends to deposit there will be no erosion until such a velocity is reached that no deposition will take place and I know of instances where that velocity was 3 ft. per sec. with no erosion.

"I have found on the inlet ditch to the Model Reservoir near Trinidad, Colo., where the water carries silt in suspension, that erosions take place in shale with a velocity of 6 ft. per sec. while in the outlet ditch where the water is clean no erosion takes place in the same character of shale with a velocity of 6 ft. per sec.

"The velocities I have given for the above soils are therefore based on a moderately clear water.

"I have made no actual determination of highest non-scouring velocities and soil analysis with a view of publishing them, but as Hydrographer and Deputy State Engineer in the State Engineer's office at Denver, Colo., I have rated hundreds of ditches located in all kinds of soils and know the velocities that cause some of them to scour and in others that do not, and as engineer and manager for different ditch systems, I have had occasion to measure velocities of water in various soils and shales and know at what velocities erosion takes place in them."

Mr. Taylor is quoted as follows:

"Fine to medium sand: The highest velocities in use through canals in this type of soil, shown by water measurements, are mean velocity, 1.7 ft. per sec. for Q equal to 45 sec-ft., and a mean, V, on tangents. The physical condition of the original soil was 60% passed through screen 74 mesh to the inch.

"At the above velocities erosion will show in places where submerged débris or weeds have lodged.

"Without having made actual tests, I think the above are the highest permissible mean velocities for the above type of soil.

"The coarse sand on the Umatilla Project through which some of the original canals were built, was so porous, and the losses were so great, they had to be lined. Some of this sand passed only 40% through screen 74 mesh to the inch."

Mr. Williams gives his experience with Wenatchee soils:

"The soil encountered on this project falls mainly into two general classifications: (*a*) heavy clay loam, and (*b*) sandy loam. The experience here has been that the first classification will stand velocity up to 3.0 ft. per sec., without serious erosion, and that the second classification erodes badly on curves at velocity of 2.0 ft. per sec. In stiff clays, coarse gravel, and shale, we have had no erosion. With very fine loam and silt, we have erosion to a considerable extent with velocity of 2 ft. per sec. The velocities on the project in unlined canal range from 2.0 to 3.0 and above observations are based solely on experience within these limits."

In addition to answers to the questionnaires, much excellent material is found in the writings of recognized authorities on irrigation engineering.

From the first great book on irrigation engineering by an American author, the late P. J. Flynn, M. Am. Soc. C. E.,* the writers quote at length (note his comment as to Dubuat and compare with the statements of the writers in the early paragraphs of this paper):

"If this slope is too great, the bed of the canal will be torn up, and the foundation of all bridges, drops and other works, will be endangered. The canal bed will be cut down and retrogression of levels take place, until the velocity of the water has adjusted itself to the cohesion of the material through which it flows. * * *

"The maximum mean velocity is not, however, so easily fixed. It must, in the first place, vary with the nature of the soil of the bed. A stony bed will stand a very considerable velocity, while a sandy bed will be disturbed if the velocity exceeds 3 ft. per sec. Some gravel beds will bear a high velocity. Good loam with not too much sand will bear a velocity of 4 ft. per sec. * * *

"In computing the slope for the Ganges Canal, Sir Proby Cantley used the formula of Dubuat. This formula was often used at this time, but is now known to be unreliable, especially for large canals.

"It is better to give too great than too small a velocity, as, in the former case, measures can be adopted to protect the side slopes, or falls can be made in the canal and the longitudinal slope, and, therefore, the velocity reduced. In the latter case the deposition of silt will necessitate an annual clearance of the canal, at great expense, and the loss of ground along the canal banks on which to deposit the spoil."

In his chapter on maximum and minimum velocities, B. A. Etcheverry, M. Am. Soc. C. E., recognizes the fact that the velocity necessary to start erosion is much higher than that required to maintain loosened particles in motion. After quoting other authorities he gives a tabulation (reproduced as Table 4) of suggested maximum mean velocities that does not permit of a direct place in Table 3, as the categories are different.

In speaking of farm ditches in particular (hence, those of shallow depth), Fortier says:†

"In fine sand or sediment a flat grade is required to prevent scouring. A mean velocity of 1 ft. per sec. is sufficient for such material. In hard gravel or hard clay or in a mixture of these, a velocity of 3 ft. per sec. can be used without eroding the bottom. In ordinary materials, ranging from sandy or

* "Irrigation Canals and Other Irrigation Works," by P. J. Flynn, San Francisco, 1892, p. 32.

† "Use of Water in Irrigation", by Samuel Fortier, M. Am. Soc. C. E., New York, Second Edition, 1916, p. 33.

gravelly loams to clay loams, a grade may safely be adopted which will produce a mean velocity of 2 to 2½ ft. per sec."

TABLE 4.—MAXIMUM MEAN VELOCITIES SAFE AGAINST EROSION.*
(By Etcheverry)

Material.	Mean velocity, in feet per second.
Very light pure sand of quicksand character	0.75- 1.00
Very light loose sand	1.00- 1.50
Coarse sand or light sandy soil	1.50- 2.00
Average sandy soil	2.00- 2.50
Sandy loam	2.50- 2.75
Average loam, alluvial soil, volcanic ash soil	2.75- 3.00
Firm loam, clay loam	3.00- 3.75
Stiff clay soil, ordinary gravel soil	4.00- 5.00
Coarse gravel, cobbles, shingles	5.00- 6.00
Conglomerates, cemented gravel, soft slate, tough hard-pan, soft sedimentary rock	6.00- 8.00
Hard rock	10.00-15.00
Concrete	15.00-20.00

* "Irrigation Practice and Engineering," Vol. II, The Conveyance of Water, by B. A. Etcheverry, M. Am. Soc. C. E., New York, 1916, p. 57.

In order to show examples of relatively high velocities in irrigation channels, Table 5 has been compiled.

Deduction from Available Data.—After a careful study of all the data presented and in the absence of experimental data bearing on the subject, the Committee recommends the values given in Table 6 for maximum permissible mean velocities. The figures given are for canals with long tangents predominating throughout their lengths. For the same canals in sinuous alignment a reduction of about 25% is recommended. Likewise, the figures are for depths of 3 ft. or less. For greater depths a mean velocity greater by 0.5 ft. per sec. may be allowed. The velocity in canals carrying water free from muddy silts, but bringing a powerful abrasive from the river, should be reduced 0.5 ft. per sec. Basalt ravelings are probably the sharpest scouring agency carried by any of the Western streams, and many of the streams from the Northwestern lava beds will contain this abrasive although the water may appear quite clear. Canals from streams more or less silt-laden through the year, like the Rio Grande or the Colorado River, may be designed for full-load mean velocities from 1 to 2 ft. per sec. greater than would be permissible for the same material or new excavation, served by clear water from a reservoir. Deep canals conveying water with clean sand will develop destructive sand boils which would be absent in the same canal with clear reservoir water. In short, the character of water conveyed must be considered as well as the materials through which the canal is first excavated. In theories of "loading", clear water is best fitted to take on a load of scoured material and a water carrying a partial load of sharp abrasive is better equipped to start erosion.

Explanation of Table 6.—The general basis of Table 6 is a tabulation of original soil materials in their order of ability to withstand erosion under usual conditions; that is, conveying relatively clear water, for the greater

TABLE 5.—EXAMPLES OF CANALS CONVEYING WATER AT RELATIVELY HIGH VELOCITIES.

TABLE 5.—EXAMPLES OF CANALS CONVEYING WATER AT RELATIVELY HIGH VELOCITIES.

Authority.	Canal.	Location.	Soil.	Flow, in second-feet.	Depth, in feet.	Mean velocity, in feet per second.	Remarks.
P. S. Flynn	Cavour	Italy	...	...	...	5	Over gravel bed.
"	Naviglio	"	...	...	...	...	...
"	Grande	"	...	...	...	5 to 6	...
"	Martesana	"	...	...	...	5 to 6	...
"	Crappone	France	...	...	...	6.6	Istres Branch.
"	Baree Doab	India	...	3 000	...	5.0	Over gravel bed.
"	Del Norte	Colorado	...	2 400	5.5	5+	Over gravel drift and rock.
"	Eastern Jumna	India	...	...	...	1.93	Perfectly adjusted to a light, sandy soil.
"	" "	"	...	...	...	2.28	Silt being deposited.
"	" "	"	Clay, shingle and small boulders	...	...	5.47	...
"	Ganges Canal	India	Sandy with admixture of clay	...	...	2.92	Bed silted.
"	" "	"	In stiff clay	...	...	4.12	Erosion trifling; no silting.
"	" "	"	In very light sand	...	...	2.50	...
S. Fortier*	Middle Creek Canal	Montana	...	...	...	7.5	Bed washed clean of all earth and sediment.
"	Bear River Canal	Utah	Clayey loam	225	...	3.62	Channel covered coating of sediment, but free from vegetation.
F. C. Scobey†	Interstate Canal	Nebraska	In clay	830	...	4.75	...
"	"	"	"	"	...	3.86	...
"	"	"	"	"	...	4.66	Bed graded material, fine gravel to 6-in. radius.
"	Salt River Valley	Arizona	Graded gravel	131.3	...	3.12	Bed scoured clean, sides silted, slick.
"	Grand Canal	"	In firm sand and gravel	161.7	...	2.72	...
"	Empire Canal	Colorado	In clay loam	372.1	...	2.94	Now has deposit of clean sand in middle, silty mud at sides.
H. D. Newell‡	D. Klamath Project	Oregon	In fine sand	155	...	1.98	
Charles Taylor‡	Umatilla Project	"	Fine to medium sand	260	...	2.4	60% of original soil passed a 74-mesh screen.
J. L. Lytel‡	Sunnyside Canal	Washington	Coarse gravel	1 260	5.4	5.25	On tangent.
"	Mabton Canal	"	Fine gravel	120	4.2	2.85	On tangent.
"	Snipes Mt. Canal	"	Fine sand	125	4.2	1.95	On tangent.
"	Rocky Ford Canal	"	Fine sandy loam	100	4.3	1.95	On tangent.
"	Snipes Mt. Canal	"	Volcanic ash	28	2.1	1.85	On tangent.
"	Mills Power Ditch	"	Alluvial silts	...	...	1.65	On tangent.
"	Snipes and Allen	"	Fine loam	...	...	1.65	On tangent.
"	Lower Richland	"	Fine sandy loam	...	...	1.00	On tangent.
"	Moxee Company	"	Volcanic ash	...	...	1.50	On tangent.
"	Old Reservation	"	Fine gravel	...	...	3.00	On tangent.
"	Yakima River	"	Coarse gravel	...	...	6.50	On tangent.
Kutter§	Mill Race	Switzerland	Side slopes, earth; bed, fine gravel	...	...	3.22	...
"	Gurbeu Canal	"	Regular reach, detritus	...	...	4.79	...

* *Water Supply Paper No. 43*, U. S. Geological Survey.
† *Bulletin 194*, U. S. Dept. of Agriculture.
‡ By letter.
§ Hering and Trautwine translation.

TABLE 5.—(*Continued.*)

Authority.	Canal.	Location.	Soil.	Flow, in second-feet.	Depth. in feet.	Mean velocity, in feet per second.	Remarks.
Kutter*	Smith Canal	Switzerland	Earth, no detritus	....	10.8	5.53	....
"	Simme Canal	"	Very coarse gravel and detritus	....	....	5.99	....
"	Escher Canal	"	Very coarse gravel and detritus	....	....	8.36	....
Fortier†	Hyrum Canal	Utah	....	45.6	1.6	3.24	....
"	Melville Canal (No. 1 Ditch)	"	....	17.4	1.3	3.34	....
"	Paradise I. and R. Co. Canal	"	....	50.3	1.5	3.13	....
"	Hooper Canal	"	....	129.5	20+	3.90	....
"	College and City Ditch	"	....	7.72	0.5	3.45	....
Cone-Trimble-Jones‡	Rio Grande	Colorado	In gravel	143.6	....	3.86	....
Cone-Trimble-Jones‡	" " Lateral 1	"	In graded gravel	38.0	....	4.66	Bed scoured clean, sides silted, slick.
U. S. Reclamation Service§	....	North Platte project, Nebraska	Sandy, fair condition.	1 154	....	2.89	Erodes, some brush rip-rap.
U. S. Reclamation Service§	....	"	Sandy, good condition.	1 096	....	3.09	....
U. S. Reclamation Service§	....	"	Gravelly, fair condition	....	....	3.32	....
U. S. Reclamation Service§ Don Bark‖	South Side Twin Falls, Main Canal	Boise project Idaho	Clay and hard-pan	....	....	3.22	In good order, no weeds.
"	South Side Twin Falls, High Line Canal	Snake River, Idaho	Dark uniform clay loam	2 737	7.53	3.43	....
" ¶	South Side Twin Falls, Farm Lateral	Snake River, Idaho	Dark uniform clay loam	877	5.31	2.72	....
		Snake River, Idaho	Medium clay loam	222	.38	2.40	....
"	Randall Canal	Snake River, Idaho	....	135.7	2.49	3.10	....

* Hering and Trautwine translation.

† Unpublished.

‡ *Bulletin 194*, Colorado Agricultural College.

§ *Reclamation Record*, July, 1913.

‖ Ninth Biennial Rept., State Engr. of Idaho.

¶ Biennial Rept., State Engr. of Idaho, 1911-12.

part of the season, either from a mountain stream or from a reservoir. In soil gradations finer than gravels, it is recognized that the resistance to erosion depends on the cohesion of individual particles, one for another. For the gravels—fine, coarse, and cobbles—the resistance is by virtue of weight, shape, and density of units, aided by the mechanical obstruction afforded one unit by its mixture with others. There is no element of cohesion due to viscosity, plasticity, or stickiness, such as is found in clay. In graded materials, loam to cobbles in size, such as are found on the Piedmont benches and alluvial fans of Colorado, Utah, and other mountain States, the canal bottoms become firmly bedded after a few years. At first, there being little plastic colloidal matter, the finer grained soils are washed free and carried farther down stream, to be deposited on the inside of curves or precipitated where velocities are reduced above check structures. At the same time, the gravel is moving slowly down stream, until finally a bed is developed in which voids are filled by smaller material, much on the order of proportioning concrete aggregates. This graded bed is quite stable, entirely because of mechanical intermixture and not because of plastic cohesion, although as indicated in Column (3) of Table 6, the fact is recognized that the admixture of effective colloids will make the bed all the more tough and tenacious, increasing its resistance to erosion. The shales are taken as smooth, soft rock, resistant to erosion by clear water, but not so resistant when a sharp abrasive, such as sand or gravel, is carried by this water. (Note comment by Mr. Bunger.)

TABLE 6.—PERMISSIBLE CANAL VELOCITIES.

Original material excavated for canal.	Velocity, in Feet per Second, after Aging, of Canals Carrying:		
	Clear water, no detritus.	Water-bearing colloidal silts.	Water-bearing non-colloidal silts, sands, gravels, or rock fragments.
(1)	(2)	(3)	(4)
Fine sand (non-colloidal)	1.50	2.50	1.50
Sandy loam (" ")	.75	2.50	2.00
Fine loam (" ")	2.00	3.00	2.00
Alluvial silts when non-colloidal	2.00	3.50	2.00
Ordinary firm loam	2.50	3.50	2.25
Volcanic ash	2.50	3.50	2.00
Fine gravel	2.50	5.00	3.75
Stiff clay (very colloidal)	3.75	5.00	3.00
Graded, loam to cobbles, when non-colloidal	3.75	5.00	5.00
Alluvial silts when colloidal	3.75	5.00	3.00
Graded, silt to cobbles, when colloidal	4.00	5.50	5.00
Coarse gravel (non-colloidal)	4.00	6.00	6.50
Cobbles and shingles	5.00	5.50	6.50
Shales and hard-pans	6.00	6.00	5.00

For each grade of original material shown in Column (1) the fact is recognized in Column (2) that the final condition of a canal bed must be quite largely determined by the material through which it flows and likewise the fact that there is little or no mechanical abrasive in the water. The

figures in Column (3) recognize that the water contains effective colloidal silts, such as the waters from the Colorado River or Rio Grande. These silts will settle to greater or less extent under relatively high velocities. The difference between colloids and colloidal silts should be noted. The first will remain in suspension, even if all velocity be eliminated, for a long period of time, even years. The silts will precipitate under reduced velocities, and possess sufficient colloidal matter to form a plastic, highly cohesive mass. Obviously, such water will have a marked influence on the final character of the canal bed for all gradations up to the shales where the original material is more resistant than any deposit of silts. In Column (4), the figures indicate that waters conveying abrasive sand or gravel will make some materials more resistant by furnishing the constituents needed for a graded bedding. On the other hand, shales or slick tough clays are themselves resistant, and this resistance is reduced when a powerful abrasive is contained in the water.

Conclusions

1.—The laws of hydraulics governing the movement of loose silt and detritus in open channels are only distantly related to the laws governing the scouring of a canal bed and are not directly applicable.

2.—The material of seasoned canal beds is composed of particles of different sizes and when the interstices of the larger are filled by the smaller, the mass becomes more dense, stable, and less subject to the erosive action of water.

3.—The velocity required to ravel and scour a well bedded canal in any material is much greater than the velocity required to maintain movement of particles of that same material before becoming bedded or that have been raveled off by higher velocities than the bed would stand.

4.—Colloids in either the material of the canal bed or the water conveyed by it, or in both, tend to cement particles of clay, silt, sand, and gravel in such a way as to resist erosive effects.

5.—The grading of material running from fine to coarse coupled with the adhesion between particles brought about by colloids make possible high mean velocities without any appreciable scouring effect.

6.—Irrigation canals may be designed for the velocity that is permissible when seasoned by age, as the demand for water grows with the age of the canal and the maximum mean velocity grows with the supply necessary to satisfy this demand.

7.—Power canals are likely to be placed under peak load as soon as feasible after completion. For this reason a more conservative velocity should be chosen, otherwise scour may take place before seasoning.

8.—Canals, when new, may be operated with velocities less than the maximum permissible, by the use of check structures.

9.—A slight excess of velocity is preferable to insufficient velocity. Check-drop structures will correct a slope causing erosion, but there is no method within reasonable cost by which velocities may be increased.

10.—In the past, engineers have been led astray by too close an adherence to the results of experimental data on the transporting action of water on such materials as clay, silt, sand, and gravel, considered separately rather than on cohesive or mechanical combinations, and in consequence many canals have been designed and built on too flat slopes with correspondingly low velocities.

11.—The growth of aquatic plants is but partially connected with velocities. Canals designed for the highest permissible velocities, from the standpoint of erosion, will be as free from plant growth as design alone can effect.

EXCESS CONDEMNATION IN CITY PLANNING

A SYMPOSIUM*

* Presented at the meeting of the City Planning Division, New York, N. Y., January 22, 1925.

THE PRESENT STATUS OF EXCESS CONDEMNATION IN THE UNITED STATES

By Frank B. Williams,* Esq.

Excess condemnation is the technical remedy for a legal technicality, and being the outgrowth of bad logic, is difficult to explain clearly. It has been defined as the taking by eminent domain (and, therefore, without the consent of the owner) of more land than is needed for a public improvement. The practical man realizes that the development of the land outside the lines of the given improvement, so as to harmonize with the improvement within them, is essential to the attainment of the maximum usefulness of that improvement. In short, he knows that the outside land—not in fact "excess"—is needed for the improvement. This, the judges in the United States, apparently have not been able to realize. The result is that the Courts in this country have held as unconstitutional several statutes of excess condemnation, and in a few States constitutional amendments have been passed authorizing it.

There are two main purposes for which excess condemnation, as it may for convenience be called, can be used with advantage in public enterprises, as follows:

1.—It may be used for the profit to be obtained by the re-sale of the "excess" land.

2.—It may be used for the purpose of controlling the development of this "excess" land so that it and the main improvement may best be utilized each in connection with the other and both render the greatest service to the community.

A few illustrations may not be out of place. Excess condemnation is usually advocated either for the laying out of a new business street, or the widening of an existing one, in a neighborhood in the center of the city where the present developments are unsuitable or unsanitary and of low value; or for the construction of a thoroughfare in the outlying part of the town where the land is relatively unimproved. In these cases one of three results may occur, as follows:

First.—The neighboring land may be increased in value, in which case it would be good business, if there is sufficient capital and the matter can be handled with skill, to take this land and sell it, to pay in part for the street. Failure to follow this course increases the cost of this and similar enterprises, to the public loss. In this age of keen competition no private business could long neglect such incidental economies and escape bankruptcy.

Second.—There may be parts or remnants of lots on one or both sides of the new street which are too small for independent development and which also shut off land from it and, therefore, from proper improvement. If the uniting of the remnant and the land in the rear is left to private initiative, considerable time may elapse, during which proper buildings will

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not be erected on the street, or cheap ones will be constructed, with a loss, in either case, to the city in taxable values and to the community in the lessened economic service of the land and the street. Often the character of a street is in this way permanently lowered, the ugly temporary structures giving it a character which it never afterward outgrows.

Third.—Even if the contiguous land is not cut into remnants, the use to which it is put may lessen the value of the thoroughfare. A boulevard with ugly houses along it, is not the beautiful boulevard for which the city has spent its money so freely. A view which the boulevard was planned to exhibit, may be cut off by a row of houses, whereas the erection of detached structures, with ample space between, would have left it open and the land would have sold as well if not better. A much needed industrial street with the adjoining land cut into small irregular lots is not capable of use for enterprises requiring large sites. The desired conditions may often be obtained by the proper subdivision and planning of the contiguous land, and its re-sale for private use without any legal restriction on it, because conditions suited to the street having once been created, may be relied on, as a rule, to continue. Of course, legal restrictions in favor of the city may be placed on the property if deemed wise; and, when suitable, such restrictions often enhance the value of the property affected, as real estate dealers and owners have found by experience.

The thoroughfare is not the only improvement with which excess condemnation may be used; indeed it may be used as a part of most public works. Thus, a new city hall properly located and constructed will cause neighboring land values to increase; and the public by proper business management could secure the profit which its enterprise has created, condemning and re-selling the land for that purpose. A beautifully designed city hall is not the structure of beauty it should be unless it has an appropriate setting. Often the cheapest and indeed the only feasible way of obtaining some measure of setting is by the control of the architecture of near-by buildings. In similar ways the construction of parks may be made a source of income to offset in part their cost, their beauty and utility being increased by the control of the neighborhood which only excess condemnation will give.

Excess condemnation, and the various expressions used for it in foreign countries, are terms of modern origin; indeed the concept itself is the product of a legalistic age. Excess condemnation in fact, however, has been practiced very generally throughout the past. A monarch, not hampered by the division of powers, has found it quite natural to take abundant land for an enterprise he had undertaken, carrying it out as seems wise, and selling any surplus or giving it away with any necessary conditions attached to insure the completeness of the work. All will remember the humble citizen in one of Molière's plays, who, suddenly and enormously enriched, took a Master of Philosophy as an instructor, only to discover to his amazement that for forty years he had been speaking prose without knowing it. Thus, the administrators of former ages used excess condemnation with entire ease before, technically, such a thing existed. It is to excess condemnation in this sense that we owe the Place des Vosges in Paris, France, and many other beautiful and useful squares and streets in Europe.

Excess condemnation in the modern meaning of the expression, seems to be French in origin, and is to be found in most foreign countries. It has been extensively practiced in England, usually for the profit that could be recouped to pay in part for the main improvement. It was provided for and considerably used in the State of New York under a statute enacted in 1812, but was held to be unconstitutional by the New York Courts in 1834, and virtually ceased to be practiced until revived by statutes in Massachusetts and Ohio in 1904 and subsequently in other States. These statutes were so criticized by the Courts that little use was made of them and constitutional amendments for the purpose were passed, first in Massachusetts (1911), and then in Ohio (1912), Wisconsin (1912), New York (1913), and Rhode Island (1916).

Whatever doubt there may have been with regard to the validity of the statutes of excess condemnation, under the State Constitutions as interpreted by the State Courts, there can be no doubt with regard to the validity of the constitutional amendments, which can be challenged only on the ground that they are contrary to the Constitution of the United States as construed by the Supreme Court. This may be asserted with confidence in spite of the fact that no case involving excess condemnation has ever come before the Supreme Court. There is an impression that the State Courts are often more narrow and technical than the United States Supreme Court. Be that as it may, this is not the reason for the confidence so generally felt by students of the subject. Land taken by excess condemnation, like land appropriated by condemnation proper, is acquired by eminent domain, which can be invoked only for a public purpose. In deciding what is a public purpose the Courts are guided both by history, which is past usage, and by current custom, which is history in the making. Local conditions also have great influence. In a country as large as the United States, past history, present opinion and custom, existing physical conditions, and surrounding circumstances, differ greatly in its widely separated sections. It cannot be assumed, therefore, that there is any one standard that can be set for the entire country, in accordance with which such questions can be decided; the locality must be studied and the question settled in the light of what it reveals. For instance, the Supreme Court has held that irrigation is a public purpose in a dry State like California, whereas it might not be so in the Eastern States. In so deciding, the United States Supreme Court followed the decision of the Supreme Court of California. The Supreme Court gives great weight in the decision of such matters to State laws and constitutional provisions as interpreted by the highest Court of the State in question, considering them to be most persuasive evidence of local conditions; so much so that in no case has the United States Court held invalid, under the National Constitution, a taking of property by eminent domain under a State law upheld by the highest Court of that State.

Excess condemnation has been utilized with success abroad, especially in England. In this country, it has been used comparatively little, in spite of the fact that the need of it, in numberless instances, is clear.

One of the reasons excess condemnation is not more used in the United States is that in many instances the procedure provided for it is not practical. City planners, in various parts of the country, although fully realizing the

advantages of excess condemnation in an enterprise, cannot see any way under existing laws to make it a success. A study of the laws of excess condemnation throughout the country, in consultation with city attorneys and engineers, would be a great service to the cause of city planning.

A second reason excess condemnation is so little resorted to in this country is the undue conservatism of city attorneys. They have an excessive reluctance to apply methods that are not already in general use, and thereby sacrifice the progress that can come only from improvement in method. The writer is also at times tempted to think that this comes in part from an unwillingness to do the work involved in any change in their usual routine.

A third reason excess condemnation is so little used in the solution of problems urgently calling for it, is the debt limit and the financial condition of cities. To take land several times in excess of that needed for the work within the narrow lines of the principal improvement calls for the outlay of many times the capital required for the main improvement; and, with many public enterprises urgently calling for funds, and a rigid debt limit almost if not quite reached, it seems impossible to get the necessary capital for the many years during which it will be tied up. A new thoroughfare, or similar undertaking, can be begun with a small expenditure, the cost being assessed against those especially benefited, to be paid by them in due course. In addition, there is the conservatism of city administrators who find it so much easier to do the work in the old way, even if thereby many of its advantages are sacrificed. There is also the unwillingness still existing among the mass of the voters, to the giving of authority to officials to enter into what is, in some of its aspects, a business transaction involving some speculative elements and the possibility of favoritism and corruption.

A fourth reason excess condemnation is not appreciated is a lack of knowledge of the advantages of city planning and the evils of unregulated growth. In the United States excess condemnation is not needed as it is abroad for the recoupment of expense to be obtained by it; in this country these advantages can be obtained by imposing local assessments on those especially benefited. In some States constitutional provisions forbid betterment assessment entirely or to a considerable extent. These constitutions can be changed, and in other States there are precedents for such amendments that have been tried and found to be sound. In some States the present statutes do not provide, adequately at least, for local assessment. Adequate legislation can be passed, with comparative ease, along lines adopted with success elsewhere. In many States the machinery exists and is constantly in use. Excess condemnation is needed for the harmonious planning and subsequent development of the land within and just outside, the lines of the main improvement; and for this there is no substitute for excess condemnation. In this country the law is often the impediment to progress. In city planning, there is the decision rendered in 1920 of the case of Windsor *vs.* Whitney by the highest Court in Connecticut. It is the so-called practical man who lags behind; and in this, as in so many other matters, the engineer who is at once a theorist and a man of affairs, has the privilege and the duty to lead the way and to receive as his reward the satisfaction of knowing that he has performed a public service.

THE USE OF EXCESS CONDEMNATION IN THE OPENING, WIDENING, AND EXTENSION OF STREETS

By A. L. Vedder,* Assoc. M. Am. Soc. C. E.

Cities have always found it necessary to open, widen, and extend streets. During the last few years, since the development of City Planning, there has been more of this activity than formerly.

When these openings, widenings, and extensions are made in developed parts of cities, irregular pieces are often left and the cost of the part taken may be as great as that for the whole lot, the claim being sustained that "the damage to the part remaining equals the value of the whole property." Sometimes the parts of lots left are not sufficient to insure a proper development.

Excess condemnation has been suggested as a cure for these ills. The writer will give the law developed and instances where it has been found to be advantageous in Rochester, N. Y.

Law.—In 1913 a new paragraph was added to Section 7 of Article 1 of the Constitution of the State of New York, which provided that the Legislature might authorize cities to take "excess condemnation". In 1921 the City Charter was amended in accordance with this constitutional amendment, the Charter, as amended, reading as follows:

"*Sec. 2.—Corporate Powers.*—The city has power to receive by gift, grant, device, bequest, purchase, or condemnation proceedings, and to hold, lease, and convey such personal estate and such real estate within or without the limits of the city as the purposes of the corporation may require; to make and lay such restrictions, negative easements, or amenities, or to receive such covenants with respect to real property within or without the limits of the city as the purposes of the corporation may require; to take more land and property than is needed for actual construction in the laying out, widening, extending or relocating parks, public places, highways or streets, provided, however, that the additional land and property so taken shall be no more than sufficient to form suitable building sites abutting on such park, public place or street; to contract and be contracted with; to sue and defend, and to be sued in any Court; to make, have, use and alter at pleasure a common seal; to have and exercise all other rights and privileges conferred upon it by law or necessary to carry out its corporate functions and duties.

The following form of ordinance was then adopted:

"By Alderman........................

"Final Ordinance No......

(Title.)

"The Common Council of the City of Rochester do ordain and determine that the following improvement is necessary and shall be made, to wit;

"Sec. 1. The Common Council hereby declares its intention to acquire the fee of the following described lands, which it deems necessary for municipal purposes, to wit:

(Description of lands actually taken for street purposes.)

"Sec. 2. The Common Council hereby declares its intention to acquire the following territory in fee absolute, and declares that it deems the same neces-

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sary for municipal purposes, to wit: For street purposes; that it determines the same to be not needed for actual construction in laying out, widening, extending or relocating of said street, and determines that same is no more additional land and property than is sufficient to form suitable building sites abutting on said street.

(Description of land taken in excess.)

"Sec. 3. The whole expense of acquiring the land above described in Sections 1 and 2, estimated at.............(the estimate of the maximum cost per front foot to the property to be assessed therefor being...........; the estimate of the minimum cost per foot being...........) to be and become due and payable in.........annual installments and assessed upon.........

(Description of assessed territory.)

"Sec. 4. The Commissioner of Public Works is hereby directed to purchase said lands hereinbefore described at a price approved by the Board of Estimate and Apportionment, and in case that said Commissioner is unable to purchase said land at a price approved by said Board, the Corporation Counsel is hereby directed to institute condemnation proceedings for the acquirement of same.

"Sec. 5. This ordinance shall take effect immediately."

All sections of this ordinance, except the second, are similar to former ordinances for the opening, widening, and extending of streets, and conform to the requirements of the City Charter. This form was later modified to allow the separate assessment of Sections 1 and 2.

Ordinances were adopted and excess condemnation used for the following purposes:

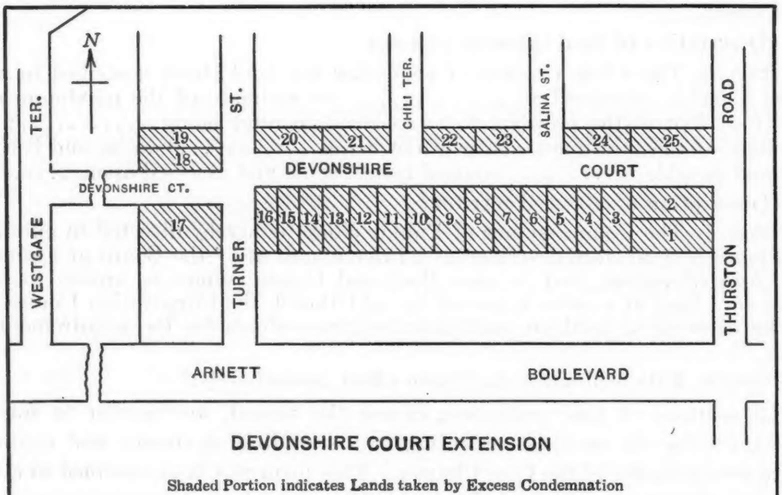
Example 1.—To Develop a Section in Conformity with a City Plan.—In 1919, the municipal authorities made a plan connecting four "dead-end" streets over a tract of vacant farm land in a moderately priced residential section. (See Fig. 1.) In 1920, an ordinance opening these streets was passed. The farm was owned by an elderly widow who asked an exorbitant price for the land required for streets, and who desired to subdivide the property in a manner different from that proposed.

If the land was to be acquired by condemnation, it was probable that a high price would be awarded by the Commissioners. The former ordinance was repealed and, in 1921, the first "excess" ordinance was passed, the land being acquired by negotiation at a rather high price.

A subdivision was surveyed and staked, and all but one lot advertised and sold at public auction, according to the provision of the City Charter. The price of the property was \$17 224; the receipts from sales amounted to \$16 750; the lot retained by the city is valued at \$2 000; thus making a profit of about \$1 500, besides opening the streets as planned without cost.

This is the only case which is near consummation and about which a financial statement can be made.

Example 2.—Reducing Cost of Opening.—A street, 80 ft. wide, connecting two existing streets in the commercial section of the city, was opened. This is part of a plan for a main thoroughfare extending from the extreme eastern to the extreme western limits of the city. This extension crossed lots diagonally and left the parts of lots not taken in an unsalable condition. (See Fig. 2.) Sufficient land was taken in excess to make good business sites on the


N
WESTGATE
TER.
ST.
TURNER
CHILI TER.
SALINA ST.
ROAD
THURSTON
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COURT
DEVONSHIRE CT.
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1
DEVONSHIRE COURT EXTENSION
Shaded Portion indicates Lands taken by Excess Condemnation

FIG. 1.

CUMBERLAND
ST.
FRANKLIN SQUARE
ANDREWS
ST.
NORTH
ST.
UNIVERSITY
AVE.
N
Dotted Lines indicate Original Subdivision
Shaded Portion indicates Lands taken by
Excess Condemnation
UNIVERSITY AVE. EXTENSION

FIG. 2.

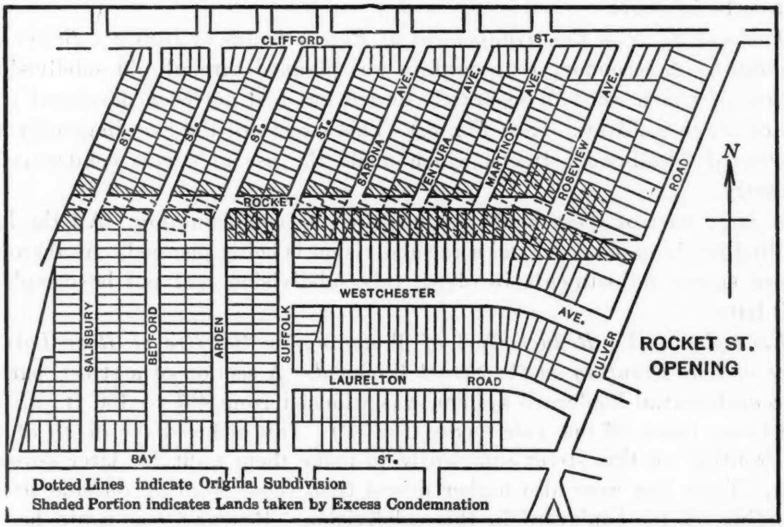
CLIFFORD ST.
SALISBURY ST.
BEDFORD ST.
ARDEN ST.
SUFFOLK ST.
SARONA AVE.
VENTURA AVE.
MARTINOT AVE.
ROSEVIEW AVE.
ROAD
N
ROCKET
WESTCHESTER AVE.
CULVER
ROCKET ST. OPENING
LAURELTON ROAD
BAY ST.
Dotted Lines indicate Original Subdivision
Shaded Portion indicates Lands taken by Excess Condemnation

FIG. 3.

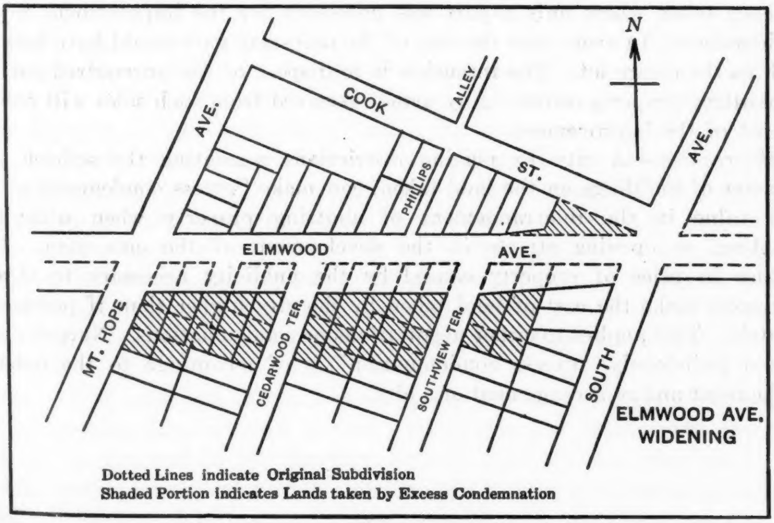
N
COOK
ALLEY
AVE.
ST.
PHILLIPS
AVE.
ELMWOOD AVE.
MT. HOPE
CEDARWOOD TER.
SOUTHVIEW TER.
SOUTH
ELMWOOD AVE. WIDENING
Dotted Lines indicate Original Subdivision
Shaded Portion indicates Lands taken by Excess Condemnation

FIG. 4.

adjoining streets. A new pavement is being laid, after which these business sites will be sold. The surrounding property having doubled in value since the property was purchased, the financial outcome of this transaction does not seem to be in doubt.

Example 3.—The Re-Arrangement of Excess Lands to Insure a Better Development.—A secondary thoroughfare was planned through old subdivisions, the lots of which were all separately owned but had not been developed from lack of sewer facilities. (See Fig. 3.) This street cut the lots diagonally and intersected "dead-end" streets, which made the use of excess condemnation necessary.

A large part of these lands was acquired by condemnation. All the land required has been obtained and a re-subdivision is being planned. As there are several survey adjustments involved, this subdivision will not be completed until later.

Example 4.—To Reduce Cost of Widening by the Use of Rear Lots in Place of Lots Fronting on the Street Widened.—A section of a street, part of a circumferential boulevard system, was widened from 49½ to 100 ft., all the land being taken off one side. (See Fig. 4.) This reduced the depth of the lots fronting on this street sufficiently to make them unfit for later development. These lots were also higher priced than those fronting on side streets extending off the boulevard in this subdivision. Recourse was again had to excess condemnation and not only the remainder of the lots fronting on the main thoroughfare, but also sufficient of those fronting on the side streets, were taken to give lots of proper depth. This area has been re-subdivided and staked.

Other Cases.—In numerous other widenings and extensions, the entire lot has been taken where only a part was necessary for the improvement under consideration. In every case the cost of the necessary part would have been as much as the entire lot. The intention is to dispose of the unrequired part to the abutting property owner. Any money received from such sales will reduce the cost of the improvement.

Observation.—A city, by placing restrictions regulating the setback and character of buildings on the land taken, can make "excess condemnation" of great value in the re-arrangement of abutting property when widening, extending, or opening streets in the development of the city plan. The increase in price of property caused by the publicity necessary to obtain ordinances make the cost of land taken by the city higher than if purchased privately. This applies tc either negotiations or condemnations. Nevertheless, if used judiciously, "excess condemnation" is of advantage to the orderly development and re-arrangement of a city.

THE NEED AND THE SCOPE OF EXCESS CONDEMNATION

By Charles Wellford Leavitt,* M. Am. Soc. C. E.

Excess condemnation! It sounds harsh and gives the feeling that it conveys no promise of benevolence. By definition it is "The lawful power, by the exercise of the right of Eminent Domain, to obtain land in a greater area than that physically necessary for a purposed public improvement, although incidental to it." The writer will discuss several reasons why excess condemnation is most necessary in present-day building of cities, and also the extent to which it should be carried, and the results of its proper use.

In opening up a new street, it is necessary:

1—To prevent waste of property.

2—To conserve to the City the greatest possible return on ratable frontage.

3—To conserve to the City its share of the increment due to the improvement.

4—To plan so that adjacent land may be replotted into lots of reasonable and useful size.

5—To protect adjoining owners from assessments unwarranted by the probable use of their lands.

Figs. 5 and 6 are views which illustrate the many small triangular areas left in private ownership by the extension of Seventh Avenue to Varick Street, New York, N. Y. About 1918, this extension was cut diagonally across a number of streets to Varick Street to form a much-needed traffic artery. Unless combined with other property, these small parcels are capable of very limited usefulness to the owner, and yet, in order to conserve to the City its needed return on frontage and the necessary increment due to the improvement created, they must be made to yield tax return. As shown, the parcels are occupied by gasoline stations and small one-story buildings. Another popular use for such land adjoining much used thoroughfares is the advertising billboard, of greater commercial value than artistic, decorative, or educational merit.

It undoubtedly will be many years before private negotiation will succeed in re-arranging property lines so that a reasonable and useful depth and frontage can be obtained on this fine new street. Instances have been cited where nearly sixty years elapsed from the time the streets were widened until the small odd-shaped lots caused by the improvements were incorporated with adjoining properties, the small parcels remaining, in the meantime, of little use and of small return. The effect on the neighborhood containing them was

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stultifying, if not directly degrading, and the City as well as the individual owners suffered rather than gained by the improvement which should have meant increased instead of depreciated values.

Figs. 7, 8, and 9 illustrate the viciousness of widening a street without contingent or excess condemnation. The narrow strips of privately owned land between the new street line and the abutting ownership, in some cases only 3 or 4 ft. wide, are too narrow to be improved by buildings—not even the odorous gasoline station can use it, and the only, but excessively apparent, use to which it can be put is that marvellous blighter of artistic hopes—the signboard.

Public interest is affected not only indirectly, but very directly, in proportion as the expense of the improved street does not bring to the City the return in ratables that it should. Both the property owner and the City are sufferers, because the property owner is assessed for benefits which he can not use, and the City cannot reasonably assess for taxes on frontage incapable of development. Many instances might be cited similar to those illustrated, not only in New York, but elsewhere as well.

Through the kindness of Charles B. Ball, M. Am. Soc. C. E., Figs. 10 and 11 are presented to illustrate the necessity for excess or remnant condemnation where diagonal streets are opened through plotted and built-up districts. The view, Fig. 12, of the unfortunate condition of a narrow lot left after widening Michigan Avenue, Chicago, Ill., is a forcible illustration of the necessity of exercising excess condemnation in such cases. Fig. 13 shows Michigan Avenue during the progress of widening and extension.

To illustrate the use of excess condemnation as exercised at Portland, Ore., the City and School District No. 1 joined forces in acquiring a site for the U. S. Grant High School and also land for adjoining playground space and for park purposes. A total of 30 acres was purchased at a price of $99 124.70, or an average $3 304 per acre for the raw land.

The profit obtained from this transaction is, as follows:

Gross receipts, 44 lots at $1 000, average		$44 000
Expenses:		
Bonded improvements	$24 800	
Brokerage, 12½% of $44 000	5 500	
Title insurance at $15 per lot	660	
Miscellaneous	440	$31 400
Profit, or refund to general fund		$12 600

The writer is not prepared to recommend this illustration of excess condemnation as he feels that it is carrying the idea beyond the limits of justification and is a trespass on the proper field of private enterprise.

An extension of Sixth Avenue, New York (Fig. 14), has been approved. The result of this plan, if carried out as shown, will be a repetition of what happened in Lafayette Street, New York, which was widened in 1903 and made a main traffic artery. Fig. 15 shows this street in 1915, still undeveloped, in a manner warranted by the expense of the improvement of widening; build-

FIG. 5.—VIEW OF TRIANGULAR AREA LEFT IN PRIVATE OWNERSHIP BY SEVENTH AVENUE EXTENSION, NEW YORK, N. Y.

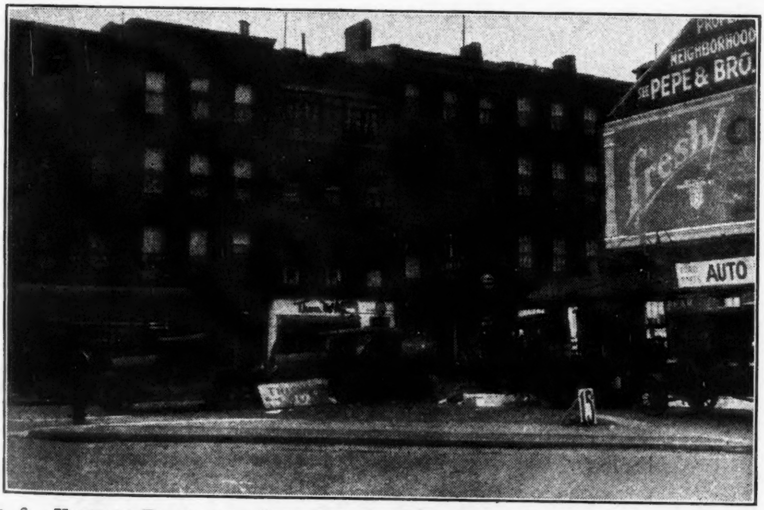

FIG. 6.—VIEW OF TRIANGULAR AREA LEFT IN PRIVATE OWNERSHIP BY SEVENTH AVENUE EXTENSION, NEW YORK, N. Y.

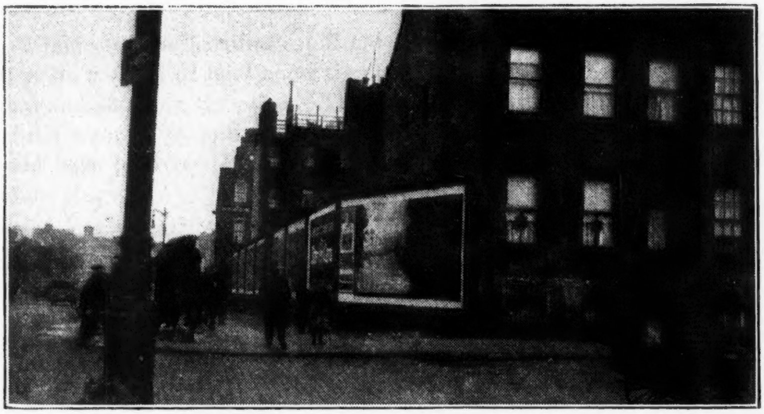

FIG. 7.—VIEW OF NARROW STRIP OF PRIVATELY OWNED LAND BETWEEN NEW STREET LINE AND ABUTTING OWNERSHIP.

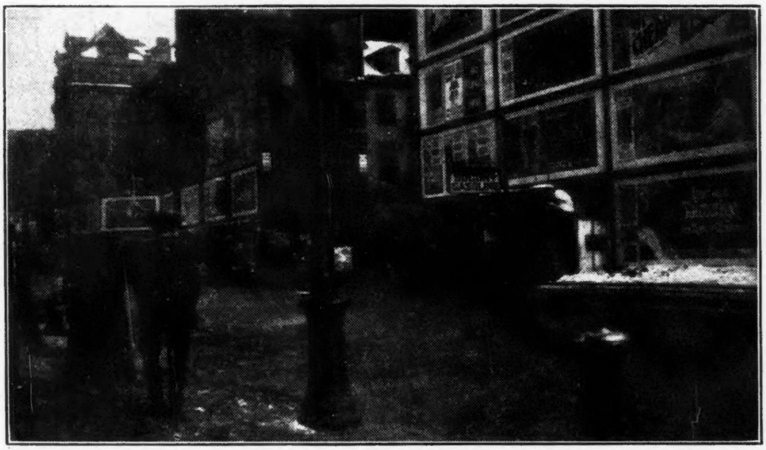

FIG. 8.—NARROW STRIP OF PRIVATELY OWNED LAND LEFT AFTER STREET WIDENING.

FIG. 9.—VIEW OF NARROW STRIP OF PRIVATELY OWNED LAND BETWEEN NEW STREET LINE AND ABUTTING PROPERTY.

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ings suitable for the location could not be built on lots 20 ft. wide and 23 ft. deep; or on a sliver of land approximately 70 ft. long, with a depth of 3 ft. on one end and 23 ft. on the other! The expedient of excess condemnation should have been resorted to, and the whole block acquired for the City, in order that it could have been re-subdivided into useful lots and sold under suitable restrictions.

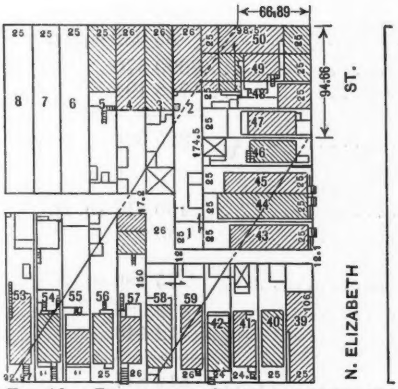


FIG. 10.—PERMANENT CONDEMNATIONS ON EXTENSION OF OGDEN AVENUE, CHICAGO, ILL.

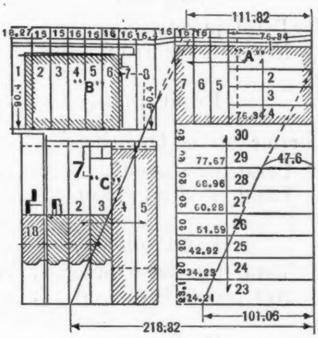


FIG. 11.—PERMANENT CONDEMNATIONS ON EXTENSION OF OGDEN AVENUE, CHICAGO, ILL.

It does not seem too strong a statement to label as a moral and economic crime, such loss to the City and to property owners. It is apparent how utterly useless small, isolated parcels of land become when left in private ownership. There is so much that can be done with the adjacent property if they are taken over by the City, and re-planned for proper development.

Fig. 16 shows the type of buildings which should be capable of construction anywhere along the Seventh Avenue Extension or on Varick Street, New York. The Grand Concourse in The Bronx is also an illustration of a fine avenue developed with suitable buildings made possible because of a land subdivision that provided lots of sufficient depth and frontage on the Parkway.

The cities of the United States are hesitating and backward about using excess condemnation. In Canada it is different; excess condemnation is exercised whenever it is deemed necessary for the public good. To illustrate, reference is made to Montreal, Que. The following figures are of interest in connection with the development of the George Etienne Cartier Square in Montreal:

Land purchased	164 504 sq. ft.
Land used as street, lots, and square............	82 426 " "
Land sold	82 078 " "
Total purchase price	$82 252
Net proceeds of sales............................	99 032
Profit	$16 780

In other words the City acquired a park which, together with the abutting street area, was about 2 acres in extent; and the profit to the City was $16 000.

The City of Montreal has also opened one street and one boulevard with the following results:

St. Lawrence Boulevard Opening:

Land taken	102 002 sq. ft.
Land used for street purposes	49 910 " "
Land sold	53 092 sq. ft.
Total purchase price	$690 570
Net returns from sales	716 194
Profit	$25 624

Cartier Street Opening:

Land purchased	130 817 sq. ft.
Land used as street	55 637 " "
Land sold	75 180 sq. ft.
Total purchase price	$99 626
Net proceeds of sale	112 443
Profit	$12 817

This method has been so successful that the Bonaparte Boulevard, which is now under construction, is being carried forward in the same manner, namely, by condemning more property than was needed, and re-selling the remainder.

In London, England, the cost of opening the following streets was reduced by the re-sale of surplus property to the extent noted:

Carrick Street	72%
Southward Street	67%
Queen Victoria Street	53%
Northumberland Avenue	$600 000 profit
Kingsway	87%

These figures were taken from the Thirty-first and Thirty-second Annual Reports of the City Park Association of Philadelphia, Pa., furnished through the courtesy of Mr. Andrew Wright Crawford.

The following is quoted from these reports:

"A recalcitrant owner might refuse to sell, except at an exhorbitant price, or even refuse to sell altogether.

"The power of excess condemnation, in addition to the power of excess purchase, which the City already has, is thus necessary to prevent the citizens from failing to get adequate results from such an improvement and to prevent the City from being mulcted. The city usually has to pay as much as an entire property is worth, where it takes half of it or more; that being so, it ought in justice to be able to condemn the entire property. In addition to this reason for excess condemnation there is the consideration, that, when this power is exercised, the owner gets absolutely all that he is entitled to. He gets the full fair value of his property at the time it is taken; and, indeed, he often gets a high value for it. When the public takes property and a

FIG. 12.—WIDENING AND EXTENSION, MICHIGAN AVENUE, CHICAGO, ILL.

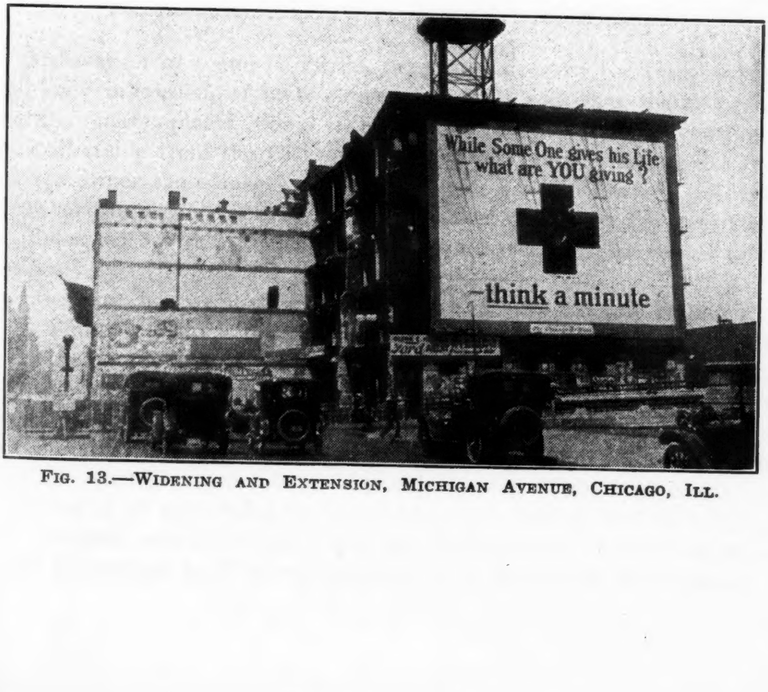
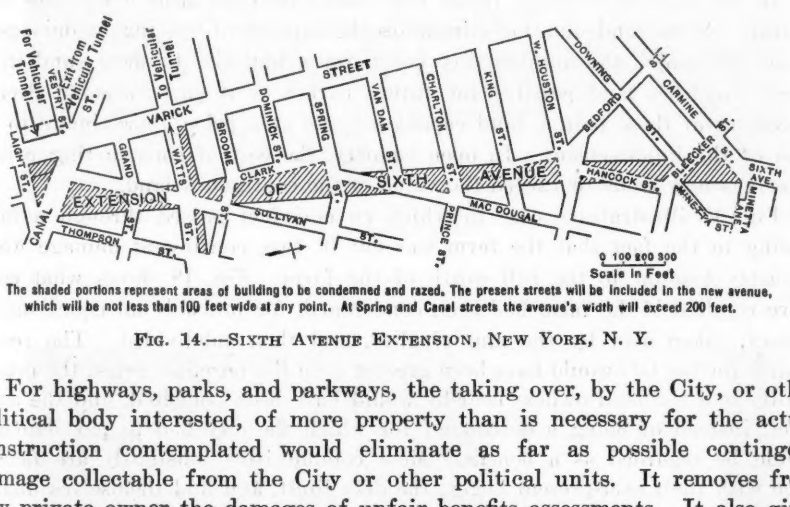


FIG. 13.—WIDENING AND EXTENSION, MICHIGAN AVENUE, CHICAGO, ILL.

jury passes upon it, the property owner is not the one who suffers; he is usually fully compensated. Even at that, most property owners want to keep as much as possible of their property, where the City only needs a portion; they realize, that if they do keep it, they will reap a harvest; because, if the City takes a third of the property for such an improvement as the Parkway, it is safe to say the City will pay at least half the value of the entire lot, and the remaining two-thirds will be worth more than the original whole lot was worth; and in the course of a few years it may double in value, owing to the improvement."

The reasons for requiring excess condemnation for street have been stated fully and illustrated. The scope of excess condemnation at the time of construction or improvement should include not only land contiguous to new and improved streets, but also land bordering on and in the immediate vicinity of highways, parks, parkways, public and semi-public building sites, sewerage systems, trunk lines, and aqueducts.



The shaded portions represent areas of building to be condemned and razed. The present streets will be included in the new avenue, which will be not less than 100 feet wide at any point. At Spring and Canal streets the avenue's width will exceed 200 feet.

FIG. 14.—SIXTH AVENUE EXTENSION, NEW YORK, N. Y.

For highways, parks, and parkways, the taking over, by the City, or other political body interested, of more property than is necessary for the actual construction contemplated would eliminate as far as possible contingent damage collectable from the City or other political units. It removes from any private owner the damages of unfair benefits assessments. It also gives to the authorities a control, not securable in other ways, over the use of land which, if deemed advisable, can be held as open spaces for the use of future generations.

Esthetics has been called the science of the suitable embellishment of practical things, of which practical things, public buildings and their sites are by no means the least. The use of the land surrounding public buildings is, in general, a subject on which in this country evidently there has not been put much clever creative thought. The creation and the preservation of fitting environments for governmental buildings, is possible only by securing not only the immediate grounds but a goodly portion of the land contiguous, the replotting of all not needed for immediate surroundings, and the selling of lots with suitable restrictions as to size and development. Excess condemnation must be resorted to, if private purchase at a reasonable figure cannot be made.

At Camden, N. J., it was recently proposed to select a site for a future civic center. After much deliberation a location was found which, although of somewhat greater area than might be required strictly for the public buildings in the group, was procurable in a single parcel. The City bought this land in total at private sale for an economical price, and now is in the position of being able to use as much of it for public purposes as may be required. The remainder can be sold under such restrictions as to preserve the environment of the civic center in a fitting manner. Also, as soon as the civic center is realized, the property adjoining will increase in value and ratables to an amount calculable only on consideration of the probable growth of the city in size and importance. What Camden has done for her future through private purchase, other cities, not so fortunate, must do by excess condemnation.

In the case of sewerage trunk lines and aqueducts near a built-up community, excess condemnation eliminates the expense of contingent damages—a cost for which the municipality has nothing material to show—and in its stead furnishes land possible for future parks; or if parks are not deemed necessary at these points, land capable of sale at a price approximate to the cost of the improvement. In open country, the cost of constructing private crossings may equal or exceed the value of the contiguous land.

Fig. 17 illustrates a case in which an aqueduct passed through a farm. Owing to the fact that the farm was cut in two, contingent damage would probably amount to the full worth of the farm. Fig. 18 shows what could have resulted if the farm had been condemned, for probably an equal sum of money, taken over by the municipality, and then subdivided. The re-sale figures for the lots would have been greater than the purchase price, the municipality and the land owners near-by would have been benefited, and the aqueduct, instead of being a detriment, for which the city had to pay damages, would be regarded as a benefit. Most communities constantly are face to face with their ever-present bogey, the debt limit, and find themselves unable, in the event of other needed expenditures, to avail themselves of the benefit of excess condemnation because of the nearly approached debt limit. Land is one of the most tangible of assets, and it seems logical that a community should be enabled to bond lands acquired under the exercise of excess condemnation, which are in excess of the true physical requirements of the purposed improvement, and that such a debt should be considered as outside the community on which limitations are placed.

This was proposed to the people of Michigan in a Constitutional Amendment, as follows:

"Bonds may be issued to supply the funds to pay in whole or in part for the excess property so appropriated, but such bonds shall be a lien only on the property so acquired and they shall not be included in any limitation of the bonded indebtedness of such municipality."

It is hoped that this idea will be realized in many places and that such indebtedness will be regarded as perfectly justifiable.

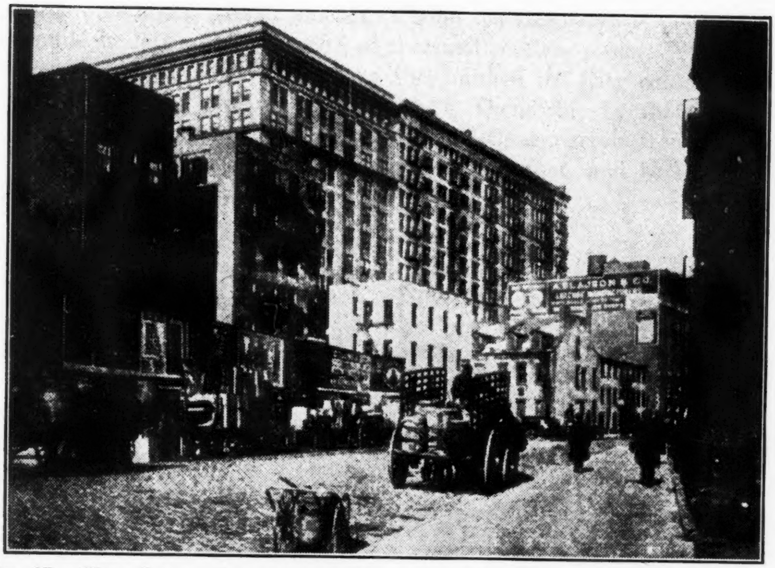


FIG. 15.—WEST SIDE OF LAFAYETTE STREET, NEW YORK, N. Y., TWELVE YEARS AFTER BEING WIDENED.

FIG. 16.—VIEW ILLUSTRATING SUITABLE BUILDINGS MADE POSSIBLE BY PROPER LAND DEVELOPMENT.

Remnant condemnation impresses one as an expression which conveys the thought of possession by the municipality of under-sized parcels of property. This would be highly proper procedure, from the viewpoint of the land owner, who would be left with possession of the small useless piece. What real use, however, would such a piece be to a City, unless the City could combine it with other property and make a good lot? Ownership by the municipality of these remnants prevents the holding up of these narrow frontages for exorbitant prices, but one should go further than that, and be sure that the

FIG. 17.

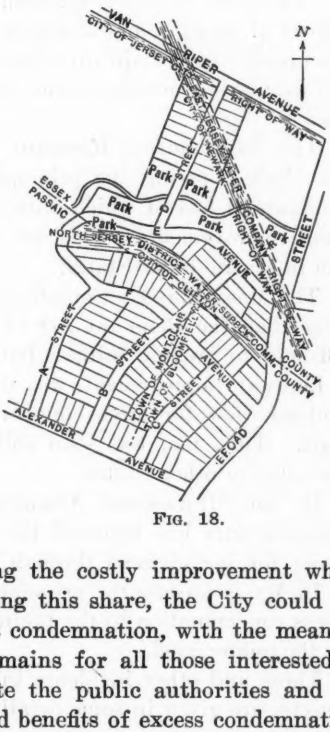


FIG. 18.

City gets its share of the profit for making the costly improvement which caused the small lots. To be safe in securing this share, the City could not take the poor end of the bargain. Remnant condemnation, with the meaning stated, is a poor half-way measure. It remains for all those interested in city planning and allied matters to educate the public authorities and the public in general, to realize the meaning and benefits of excess condemnation. How it works to the benefit of the average citizen, and enthusiastically to those in the real estate business, is self-evident because public economics mean lower taxes, the man who pays the taxes prefers to live where taxes are low, and the real estate man finds his best market there.

EXCESS CONDEMNATION IN MASSACHUSETTS

By Frederic H. Fay,* M. Am. Soc. C. E.

Synopsis

The story of excess condemnation in Massachusetts, as in most States, is a story of small accomplishment so far as the actual exercise of this power is concerned. In certain directions, however, Massachusetts has been a pioneer in formulating principles and establishing the right to use excess condemnation.

The Massachusetts Remnant Act of 1904 was the first legislation passed by any State applying the principles of excess condemnation to the taking of remnants of lots of which only portions are actually required for purposes of street improvements. This was followed shortly afterward by similar legislation in several other States.

The Massachusetts Constitutional Amendment of 1911, which is broader in scope than the Remnant Act of 1904, and permits the taking in fee of land sufficient to provide suitable building lots on both sides of a projected street or highway improvement, was the first recognition of the principle of excess condemnation for street improvement purposes in the Constitution of any State. This, too, was soon followed by constitutional amendments of like character in other States.

By the "Homestead Amendment" to the Constitution, adopted in 1915, Massachusetts has assumed the right to go into the business of providing homes for her citizens through State action.

In Massachusetts the principle has been established and upheld of applying excess condemnation to the taking of restrictive easements in private property for the public good.

These and other incidents in the story of excess condemnation in Massachusetts are given in some detail in this paper.

Excess Condemnation Applied to Land Reclamation

About seventy-five years ago, the State of Massachusetts undertook extensive work of land reclamation by a method which, although perhaps not excess condemnation in the strict interpretation of that term, is at least on the border line. This project was the reclamation of tidal flats along the Charles River, which now constitute the well-known Back Bay District of Boston. Although in most tide-water States, ownership of tidal flats rests in the State up to the high-water line, in the State of Massachusetts as well

* (Fay, Spofford & Thorndike), Boston, Mass.

as in the State of Maine, owners of water-front property own the tidal flats to low-water mark provided the low-water mark is not more than 100 rods from the shore. Title to all flats beyond the 100-rod limit rests in the Commonwealth. This private ownership of tidal flats results from an ordinance passed in 1641 under authority of the Colonial Charter of the Massachusetts Bay Colony.

The extensive tidal flats which have since been developed as the Back Bay Section of Boston were separated from the main "tidal-estuary" portion of the Charles River about a century ago by the building of a tide-mill dam on what is now the line of Beacon Street. Prior to 1850, the population of the City of Boston and of the adjoining Town of Roxbury had grown to such extent that the drainage from these communities into this tidal-flat basin had created a nuisance. Shortly after 1850, the Commonwealth undertook the reclamation of these flats as a measure of sanitation. As the simplest method of procedure, the State acquired title to the area by right of eminent domain. The flats were filled and proper drainage was provided. Suitable streets and public spaces were laid out, certain portions of the lands were granted by the Commonwealth to educational institutions, and the remaining lots were sold to private parties. The reclamation of this considerable territory was of benefit not only to the inhabitants of Boston and neighboring communities in the abolition of a nuisance, but the net result financially was a substantial profit to the State. Since that time other tidal flats along different portions of the water-front of Boston Harbor have been and are being similarly reclaimed by State agency.

Viewed from the standpoint of the abolition of a nuisance the reclamation of the Back Bay flats was not excess condemnation for the reason that the State took by eminent domain only such land as was necessary to fill in order to get rid of the nuisance. Considered as a land-reclamation project it is to be noted that the State took by eminent domain the fee in private property, filled the land, laid out streets, etc., and then in the case of a considerable portion of this land sold the fee to private parties other than the original owners. Possibly this project might have been carried out by condemning merely easements in the privately owned flats. The taking of the fee was certainly a more direct and satisfactory method, as it permitted the replotting of the land according to a well-considered plan. Probably this materially enhanced the value of the properties which were sold.

Excess Condemnation of Lot Remnants

Early Special Legislation.—The State of Massachusetts long ago passed certain special legislation dealing with the problem of remnants of land resulting from the laying out of street improvements. In 1865, an Act relating to the widening of Oliver Street, Boston (Chapter 159) was passed, which provided that any person owning an estate, part of which is taken for street uses may, "at any time before the estimation of damages * * * elect to surrender all of said estate to the city of Boston." Any remaining portion of an estate not used may be sold and "the net proceeds thereof shall be applied toward the expense of said widening and grading." This was not excess

condemnation, as the City had no discretionary power; the choice rested with the owner of the land whether or not the whole of his estate should be taken.

In 1866, another Act was passed by the Massachusetts Legislature (Chapter 174) entitled "an act concerning the laying out, altering, widening and improving the streets of Boston." Although the former Act of 1865 applied only to one particular street, this Act of 1866 was of broader scope and was intended to be general in its application as far as street improvements in Boston were concerned. Like the Special Act of 1865, this Act gave the owner of an estate the choice of surrendering the whole estate instead of the part to be taken for street improvements, provided his decision were made prior to the award of damages. Should he elect to surrender his entire estate, however, the City was not compelled to take it but could do so "if said Board of Aldermen shall then adjudge that public convenience and necessity require the taking of such estate." This may be considered excess taking by mutual consent, and it is not excess condemnation.

Massachusetts Commission on Excess Condemnation.—About twenty-five years ago the subject of excess condemnation became of much interest in Massachusetts. In 1903, the Legislature passed a resolve (Resolves, Chapter 86) "to provide for the appointment of a committee to consider the matter of making public improvements under a more extensive exercise of the right of eminent domain than is now authorized by the constitution and statutes". The Special Commission created under this Resolve went abroad and made extended investigations of the workings of excess condemnation in Europe. It reported* the results of its investigation with its recommendations to the Legislature of 1904. The Commission submitted with its report a draft of a proposed statute embodying its recommendations in part and providing for the condemnation and replotting of remnants. Dr. Cushman, in his admirable book,† which is the first work on this subject to be published in the English language, states:

"There are five features of this proposed law which are worthy of note. First, it gave cities the power to condemn unusable remnants of land. In the second place, it authorized the city to offer such a remnant to the owner of the adjoining parcel with which the public authorities deemed it wise to unite the remnant at a price fixed by an elaborate system of appraisal. Third, if this offer was not accepted the city might condemn the whole or any portion of the property of such adjoining owner. Fourth, all the surplus land taken under the provisions of the Act should be sold at public auction. Fifth, the owner of a remnant the area of which was not more than one thousand square feet might compel the city to purchase it."

A sixth point, not mentioned by Dr. Cushman, is also of interest, namely, the draft provided that after the filing in Court of the plan and petition of a proposed improvement no damages should be awarded for any building thereafter erected upon the land in question; this feature had been used in connection with street improvements in London, England, to prevent buildings being erected or improvements to buildings being made for the sole purpose of

* For Report, see House Document 288, and for Supplemental Report, House Document 1096.

† "Excess Condemnation", by Robert Eugene Cushman (National Municipal League Series), 1917, p. 62.

enhancing the damage award collected from the city, but its constitutionality in America may be seriously questioned.

Dr. Cushman states further of the statute proposed by the Special Commission:

"Such a law would allow a municipality to condemn not merely remnants of land but enough additional land to make possible an adequate replotting of such remnants into suitable building lots. The Legislature was unwilling to go to this extent, however, and contented itself with passing, in 1904, an act allowing cities to condemn merely the remnants of land left by the construction of an improvement."

The Massachusetts Remnant Act of 1904.—The Remnant Act of 1904 (Chapter 443) is an elaborate act of thirty sections and defines procedure in minute detail. Section 2 of the Act is most significant and reads as follows:

"The Commonwealth, or any city in the Commonwealth so far as the territory within its limits is concerned, may in the manner hereinafter set forth, take in fee by right of eminent domain the whole of any estate, part of which is actually required for the laying out, alteration or location by it of any public work, if the remnant left after taking such part would from its size or shape be unsuited for the erection of suitable and appropriate buildings, and if public convenience and necessity require such taking."

The term "public work" in the Act is defined as meaning any public highway, square, open space, public playground, or park. As Dr. Cushman states:*

"It is important to note that this law does not confer upon the city or State unlimited discretion in the taking of remnants. The land which may be condemned in excess of actual need is only that which is not suitable for independent development. In other words, no broad power seems to be given to condemn all estates, any part of which, however small, may be needed for actual public use."

The Massachusetts Remnant Act of 1904 was the first important Act in the United States providing for the excess condemnation of remnants of land. It was followed by similar Acts in Ohio, Virginia, Connecticut, Pennsylvania, Maryland, Wisconsin, and New York. Thus, in this field of excess condemnation, Massachusetts was a pioneer. The constitutionality of the Massachusetts Remnant Act of 1904 was upheld by an opinion of the Supreme Court of the State, rendered March 4, 1910 (Opinions of Justices, 204 Mass., 616 *et seq.*). In passing on the constitutionality of certain other legislation then pending in the Legislature, the Massachusetts Supreme Court had occasion to refer to the Remnant Act of 1904, as follows:

"We are asked whether it would make any difference if the proposed statute contained provisions like those of the statute of 1904, chapter 443, section 6. In our opinion, given to the House of Representatives, we intimated that this statute is constitutional. In our judgment it goes to the very verge of constitutionality. The grounds on which we are inclined to sustain it have little relevancy to the stated purpose of the unusual provisions of the proposed statute. They are: First, that there can be no taking outside the location of the public work except of a remnant of an estate a part of which is actually required for the laying out, alteration, or location of the public work, and then only if the remnant left after taking such part would, from its size or shape, be 'unsuited for the erection of suitable and appropriate buildings'—in other

* "Excess Condemnation", p. 63.

words, only when there is a remnant that is too small or too ill shaped to be of any practical value for the use to which valuable land is commonly put; and, secondly, that such a remnant can be taken only upon an adjudication that public convenience and necessity require the taking. Unless it can be said that public convenience and necessity never can require the taking of such a remnant the statute cannot be declared unconstitutional. While it is plain that a city or town cannot take land outside a public work for speculative purposes, we can conceive of a remnant of an estate, a part of which is necessarily taken, which remnant is so small, or of such shape and of so little value, that the taking of it in the interest of economy or utility, or in some other public interest, may be fairly incidental and reasonably necessary in connection with the taking of land for the public work."

The authority conferred by the Massachusetts Remnant Act of 1904 was seldom used. The only instance known to the writer was in the City of Springfield, where it was utilized on five occasions both for new street layouts and for the widening of existing streets and is understood to have been found of advantage to the city and protective to property interests.

The 1904 Remnant Act was repealed by the Legislature in 1918 (Chap. 257, Sec. 203 of Acts of 1918), partly because it has been little used and more particularly because under the Constitutional Amendment of 1911 all the powers of the 1904 Act and much broader powers of excess condemnation could be conferred at any time by special legislation.

Constitutional Amendment of 1911 Applying Excess Condemnation to Street Improvements

Proceedings Leading to Proposal of Constitutional Amendment.—The limited power incurred by the Remnant Act of 1904 was brought out as the result of a report of the Joint Board on Metropolitan Improvements made to the Legislature in 1910. This Joint Board, consisting of four State Commissions, created by legislative resolve (Chapter 113, Resolves of 1909) to consider the question of needed improvements in Metropolitan Boston advocated the building of an important new thoroughfare through the down-town portion of the city. This thoroughfare would not only provide an important artery for surface traffic, but would also be utilized as a route of a railroad tunnel, connecting the North Station and the South Station, the two passenger terminals of Boston. The building of this thoroughfare which would cut through closely built-up territory, would necessitate the taking of the whole or portions of more than 150 parcels of property. If the City were to proceed under the Remnant Act of 1904 and condemn in addition to the land actually required for street purposes only such lot remnants as were not suitable for the erection of proper buildings, it was felt that the results would by no means insure suitable development along the new street. The Joint Board was of the opinion that this was a case in which it seemed desirable to exercise broader powers of excess condemnation and to take, in addition to the remnants which might be condemned under the 1904 Act, other parcels which could be combined with the remnants and the whole replotted into suitable building lots.

The Joint Board reported:

"A thoroughfare such as suggested will fall far short of its full usefulness if it furnishes simply a convenient route for traffic. It cannot entirely fulfill

commercial requirements unless individuals and corporations desiring to erect important business and office buildings or warehouses and manufacturing plants can secure suitable lots thereon."

Commenting on the unsuitability of remnants remaining after takings under the Remnant Act of 1904, the Board reports further:

"If, however, takings in addition to these remnants can be made wherever needed to provide areas sufficient for the construction of buildings of large size, then the street will more quickly and surely become an important business thoroughfare, and the commercial and industrial growth and prosperity of the city will be promoted in a larger and more adequate way."

In accordance with the recommendation of the Joint Board, legislation was proposed in 1910 to broaden the powers of excess condemnation conferred by the Remnant Act of 1904 so as to make possible more extensive property takings for the new thoroughfare. In accordance with Massachusetts practice the Legislature sought and secured from the Supreme Court an advance opinion as to the constitutionality of the proposed measure, if it became a law. The Court held that the measure proposed would be unconstitutional, although by indirection it upheld the Remnant Act of 1904, which the Court believed went to the very limits of constitutionality. This decision has been reported* as follows:

"39. Opinion of the Justices, 204 Mass., 607, 91 N. E. 405, 27 L. R. A. (N. S.)—483. The House of Representatives passed an order requiring the opinion of the Justices of the Supreme Judicial Court upon the following question (in substance): If the commercial interests and general prosperity of the inhabitants of the Commonwealth and particularly the city of Boston are dependent upon the existence in that city of facilities for the transaction of foreign and domestic trade and commerce, and chief among such facilities is a broad thoroughfare with adequate sites upon it for warehouses and other buildings, and no such street exists, and if such a street should be laid out in the ordinary way the adjoining land would be left divided into parcels of such unsuitable size and shape, that the proper facilities could not be furnished by the mere laying out of a street, but could only be secured by the concentration through the exercise of eminent domain of estates abutting thereon into parcels of suitable size and shape, is it within the constitutional power of the Legislature to authorize the taking by the city of land for such a thoroughfare and of so much land on both sides thereof as may be reasonably necessary to furnish the proper facilities and with a view to subsequent use of such land by private individuals in such manner as to secure the public interests referred to?

"The answer of the Court was in part as follows:

"'The question seems to relate particularly to the power of the Legislature to take and use land outside of the proposed thoroughfare, for purposes which have no direct relation to the construction or use of the street for travel. It is presented upon the hypothesis that the desired facilities for the profitable use of the land can be secured only by the obliteration in whole or in part of the present lines of individual ownership along the street, the concentration, through the exercise of the power of eminent domain, of these abutting estates, in parcels of suitable size and shape, and the development or use of such parcels for warehouses, mercantile establishments and other buildings suited to the demands of trade and commerce. The question is whether such land can be taken with a view to the subsequent use of it by private individuals, under conveyances, leases or agreements which shall embody suitable contracts for

* "The Law of Eminent Domain", by Philip Nichols, 1917, p. 179.

the construction on the land of buildings adapted to use in domestic and foreign trade and commerce, and for the use, management, and control of the lands and buildings in such manner as to secure and promote such trade and commerce. The proposed legislation to which the inquiry relates, necessarily would contemplate action by the city in the procurement, management and control of land along a street within the city, for no other purpose than to induce and promote a use of it by merchants or traders. It would contemplate a taking of private property in the exercise of the right of eminent domain, and an expenditure of money to pay for it and fit it for occupation. It is a rule of law universally recognized in this country that neither of these things can be done unless the taking or expenditure is for a public use. This has been stated so often, and the principles on which it is founded have been considered so fully that it is unnecessary to discuss it or to cite authorities. The only question about which there is a possibility of doubt is whether the proposed use of the land outside of the thoroughfare is a public use. It is plain that a use of the property to obtain the possible income or profit that might enure to the city from the ownership and control of it would not be a public use. The city cannot be authorized to take the property of a private owner for such a purpose, nor can the city tax its inhabitants to obtain money for such a use. It could as well tax them to raise money to carry on any other private business with a hope of gain. Such proceedings are entirely outside the functions of a state or of any subdivision of a state. It is equally true and indubitable that a management and use of such property to promote the interests of merchants and traders who might occupy it, and to furnish better facilities for doing business and making profits, would not be a public but a private use of the real estate. An affirmative answer to this question would make it possible for the city to take the home of a resident near the line of the thoroughfare, or the shop of a humble tradesman, and compel him to give up his property and go elsewhere, for no other reason than that, in the opinion of the authorities of the city, some other use of the land would be more profitable and therefore would better promote the prosperity of the citizens generally. We know of no case in which the exercise of the right of eminent domain or the expenditure of public money has been justified on such grounds. * * * We answer the question in the negative.'"

Constitutional Amendment of 1911.—Finding in the case of the proposed law for the construction of a new thoroughfare between the North and South Stations, that the exercise of broad powers of excess condemnation was not possible under the Constitution of Massachusetts as it then existed, the Legislature of 1910 proposed a constitutional amendment as follows:

"Article ten of part one of the constitution is hereby amended by adding to it the following words: The Legislature may by special acts for the purpose of laying out, widening or relocating highways or streets, authorize the taking in fee by the Commonwealth, or by a county, city or town, of more land and property than are needed for the actual construction of such highway or street: *provided, however,* that the land and property authorized to be taken are specified in the act and are no more in extent than would be sufficient for suitable building lots on both sides of such highway or street, and after so much of the land or property has been appropriated for such highway or street as is needed therefor, may authorize the sale of the remainder for value with or without suitable restrictions."*

In Massachusetts a constitutional amendment must be passed by two successive Legislatures and then ratified by the people. Accordingly, the amendment quoted, passed by the Legislature of 1910, was again passed by the

* Now Article XXXIX of the Constitution of Massachusetts.

Legislature of 1911 and was ratified by the people at the election on November 7, 1911.

It is to be noted that the Constitutional Amendment of 1911 relates solely to the laying out, widening, or relocating of highways or streets. Also, that the powers of excess condemnation made possible by this amendment may only be exercised through special acts of the Legislature.

Massachusetts was the first State to adopt a constitutional amendment of this character. It was followed by amendments in Ohio and Wisconsin in 1912, in New York in 1913, and in Rhode Island in 1916.

Special Legislation Under Provisions of 1911 Constitutional Amendment.— Several Special Legislative Acts have been passed under this constitutional amendment although the project for the proposed thoroughfare between the North and South Stations in Boston was not carried out. Among the Special Acts may be noted the following:

Acts of 1912, Chapter 186, authorizing the City of Worcester to take land for the widening of Belmont Street.

Acts of 1913, Chapter 169, relating to highway improvements in the City of Brockton.

Acts of 1913, Chapter 201, authorizing land takings by the City of Worcester in connection with the extension of Madison Street.

Acts of 1913, Chapter 326, authorizing the City of Worcester to take certain lands near Washington Square and to sell part of the same.

Acts of 1913, Chapter 703, providing for the widening of Bridge Street in Salem.

As far as the writer is aware, the authority to use excess condemnation, conferred by these several Special Acts, has in no instance been exercised.

It is to be noted that the constitutional amendment permits special legislation to be passed authorizing the sale of remaining land with or without restrictions. The permission to sell remaining land with restrictions was granted in some but not all of the Acts enumerated. There seems to be some doubt whether a city could impose restrictions unless expressly authorized so to do by legislation or by the State Constitution.

The only Special Act permitting the application of excess condemnation, which has been passed with relation to street improvements in the City of Boston, is that relating to the Stuart Street Improvement (Special Acts of 1917, Chapter 329, as amended by Special Acts of 1918, Chapter 118; Acts of 1920, Chapter 312; Acts of 1920, Chapter 465; Acts of 1921, Chapter 407). The Stuart Street extension was the first stage in a comprehensive plan for a "western artery" proposed by the City Planning Board in 1917 to provide an adequate street outlet for the down-town business section of the city. Under the original Act the excess condemnation plan was to be used and the Act specified the parcels of land to be taken. The City was authorized to issue bonds outside the debt limit to the amount of \$4 000 000, but was directed to enter into agreements with, or take guaranties from, property owners so that the total expense to the City would not exceed \$250 000. This requirement was due to the fact that the Park Square Real Estate Trust, which had extensive property holdings in the vicinity, gave assurance at the time the legislation was passed that it would take over the excess land on terms such

that the net expense to the city would not exceed the amount stipulated in the Act. The original Act was approved May 14, 1917, shortly after the United States entered the World War. Under war conditions it was deemed inexpedient to proceed with the Stuart Street Improvement and the Special Act of 1918 extended the earlier legislation by providing that the 1917 Act "shall take effect upon its acceptance by the Mayor and City Council of the City of Boston at any time before the expiration of one year after the termination of the present war, as defined by Federal authority." The Park Square Real Estate Trust made an offer to the City to take over the properties in excess of the actual requirements for the street on terms such that the estimated net cost to the City would not exceed the limit of $250 000 prescribed by law. This offer held until February 1, 1920. Meantime, the City Council had not accepted the 1917 Act and doubt was expressed as to the constitutionality of its excess condemnation provision which had never been tried in so large and important a project. The City Council being apparently hostile to the undertaking, no agreement was reached with the Park Square Real Estate Trust, and the excess condemnation proposal fell through. In 1920, further legislation was passed authorizing the Stuart Street Improvement without the excess condemnation feature, and the street has since been constructed without the exercise of excess condemnation.

One other proposal for the extensive application, under the 1911 Constitutional Amendment, of the principle of excess condemnation in Boston is worthy of notice. This was the Bill (Senate No. 268) introduced in the Legislature of 1920 for the North End Improvement. The section of Boston known as the North End is one of the oldest portions of the city and two centuries ago was the leading residential section. Originally, its house lots were large and the street system which was gradually developed resulted in irregular city blocks of large size. During the latter part of the Nineteenth Century the North End became a tenement house district, all available land was built upon, and many tenements were constructed in the interiors of these blocks without suitable street access and without adequate provision for light, air, and recreation. Thus, a population of great density, in certain blocks exceeding 500 persons per acre, came to be housed under unwholesome conditions. The remedy proposed was the cutting of new streets through the larger and more congested blocks and the widening and improvement of existing streets and alleys. To carry out the improvement properly, and secure suitable replotting of the land, the proposed legislation provided for a liberal amount of excess condemnation. The measure was strongly endorsed by public and semi-public organizations, was passed to be engrossed by the Senate, but was finally defeated in the House of Representatives. Thus was lost another opportunity to apply the excess condemnation principle in Boston on a large scale and to a major improvement which could not be carried out adequately by other means.

Although Massachusetts by its Constitutional Amendment of 1911 has made possible the application of excess condemnation to street improvements, this authority has not yet been exercised.

Excess Condemnation for Park Purposes Declared Unconstitutional

One instance has occurred in Massachusetts where the attempt to apply the principles of excess condemnation to land takings for park purposes has been declared unconstitutional. In 1912 the Legislature passed an Act (Chapter 715) "to make Salisbury Beach a public reservation and to establish the Salisbury Beach Reservation Commission." This Act authorized the Commission which it created to take by eminent domain any or all of certain shore lands in the Town of Salisbury within certain wide limits which were defined in the Act. The Act provided (Section 10) that the Commission "may sell or lease any lands or rights in land taken or acquired by it, which are not needed as a public reservation for the use, exercise, and recreation of the inhabitants of the Commonwealth, with or without restrictions as to its use as it may deem advisable." The Act was of such broad scope that the powers of the Commission to condemn and re-sell land were practically unlimited. Its constitutionality was brought in question before the Supreme Court in 1913. In its decision (Opinions of Justices, 215 Mass. 371), the Court held the Act unconstitutional on the ground that it was "an attempt to authorize the exercise of the right of eminent domain in part for a private purpose, and because it is impossible to separate the valid from the invalid portions of the statute," and, further, that "the interest of the public that the people should be well housed although proper to be considered in the exercise of the police power, is not a proper subject for the exercise of eminent domain." Under the takings contemplated an existing seaside resort with a number of summer cottages would have been included in the reservation.

According to Chief Justice Rugg, who wrote the decision:

"The underlying objection is that the main end of legislation for this purpose is a private utility rather than the general good. While incidentally it may be an advantage to the public that private persons prosper, if the essential character of the transaction in its direct object is private benefit to individuals, the purpose is not public. * * * Legislation which is designed or which is so framed that it may be utilized to accomplish the ultimate result of placing property in the hands of one individual for private enjoyment after it has been taken from another individual avowedly for a public purpose is unconstitutional."

In the opinion of the Court the power conferred on the Commission was too broad. It is implied in the decision that if the power to take land and sell it had been restricted as to time, or by "changed conditions", or to the sale of remnants, that part of the Act might be considered as constitutional.

It is to be noted that this opinion of the Supreme Court of Massachusetts was rendered in 1913, prior to the adoption of the Homestead Amendment to the Constitution in 1915, which does permit the Commonwealth to condemn land for housing purposes, and, after improvement, to sell it for the purpose of relieving congestion of population and providing homes for citizens.

The "Homestead" Constitutional Amendment of 1915

By 1911, prior to the shortage of homes resulting from the World War, the question of proper and adequate housing of its citizens received the attention of the Massachusetts Legislature. In that year the Massachusetts

Homestead Commission was created by legislative enactment (Acts of 1911, Chapter 607). The Commission was directed to make an investigation and report and to recommend legislation "embodying a plan and the method of carrying it out whereby, with the assistance of the Commonwealth, homesteads or small houses and plots of ground may be acquired by mechanics, factory employees, laborers, and others, in the suburbs of cities and towns." In its report to the Legislature in 1912 the Commission recommended that part of the unclaimed deposits of savings banks which had been called into the State Treasury should be loaned to the Commission for the purpose of providing workingmen's homes as contemplated in the Legislative Act of 1911.

Although the Homestead Commission Bill, as it was called, was pending in the Legislature in 1912, that body requested the opinion of the Supreme Court on the constitutionality of the measure. The Court held that the legislation proposed would not be constitutional. The main questions decided by this opinion were that the purpose of the bill was to enable the State Commission to do private business, utilizing public funds, therefore, it was unconstitutional; that the Commonwealth could not raise money for such purposes by taxation; and that money from unknown owners deposited in the State Treasury by savings banks was held in trust by the Commonwealth, subject to the claims of the owners at any time (Opinions of Justices 211 Mass. 624).

After the Supreme Court declared the proposition unconstitutional the Legislature further instructed the Homestead Commission (Chapter 714, Acts of 1912) to:

"Continue its investigation of the need of providing homesteads for the people of the Commonwealth and its study of plans already in operation or contemplated elsewhere for housing wage-earners * * * and * * * recommend such legislation as in its judgment will tend to increase the supply of wholesome homes for the people."

The Commission made exhaustive studies of governmental aid to housing in twenty-seven foreign countries, including Australasia, the United Kingdom, Continental Europe, and parts of South America and of South Africa. As a result of its studies, the Commission concluded that "if there is to be created a sufficient supply of wholesome homes within the means of the ordinary wage earners, cities and towns must be built according to well considered plans." Accordingly, two bills were recommended to the Legislature of 1913, one providing for the establishment of local planning boards by the cities and towns of Massachusetts and the other for studies by the Commission of questions of town and city planning and for the Commission's aid to cities and towns in planning work. These bills were passed as Chapter 494 and Chapter 595, respectively, of the Acts of 1913.

The first of these two Acts made City Planning Boards mandatory in every city of the Commonwealth and in every town having a population of more than 10 000 and marks the beginning of organized official city planning work in Massachusetts. This was the first mandatory city planning legislation to be passed by any State.

Being unable to find a way to provide homes for working people through governmental aid under the provisions of the State Constitution as it then

existed, a constitutional amendment was passed by two successive Legislatures, in 1914 and 1915, and ratified by the people on November 2, 1915.

The Homestead Amendment, as it is called, is as follows:

"The General Court shall have power to authorize the Commonwealth to take land and to hold, improve, sub-divide, build upon and sell the same, for the purpose of relieving congestion of population and providing homes for citizens: *provided, however*, that this amendment shall not be deemed to authorize the sale of such land or buildings at less than the cost thereof."

Two Legislative Acts have been passed upon recommendation of the Homestead Commission under the provisions of the Homestead Constitutional Amendment of 1915. One Act (Chapter 185, General Acts of 1916) permits cities to maintain schools of agriculture and horticulture. This was passed as an educational measure, as the Homestead Commission thought that:

"Great waste would result, and possibly danger to the homestead movement, if many persons, inexperienced in the care and management of the soil, were put in possession of 'small houses and plots of ground', as contemplated in the Act under which the Homestead Commission was created."

The other Act (Chapter 310 of the Acts of 1917) authorized the Homestead Commission to provide homesteads for citizens, and carried a small appropriation of $50 000 for inaugurating the work. A site of about 7 acres was purchased by the Commonwealth in the City of Lowell and a number of houses were built at moderate cost. These workingmen's homes are being sold on a long-term basis at prices ranging from $2 400 to $3 100 each. This marks the entry of the State of Massachusetts into the business of housing. Although the housing experiment at Lowell is working out satisfactorily and the State will, in due time, recover its investment, there has as yet been no disposition on the part of the State to embark upon another housing development.

As in the case of the reclamation by the State of tidal flats, noted previously, it is a question whether the "Homestead Amendment" falls under excess condemnation as strictly interpreted.

Excess Condemnation Applied to the Taking of Easements in Property

In 1893, the Legislature passed an Act (Chapter 462) granting to cities and towns authority to establish building lines parallel to, and not more than 25 ft. from, a highway or street and to prohibit the erection of buildings within such restricted area. The right was given owners to recover damages in the same manner as in the taking of land for a highway.

Supplementing this legislation of 1893, an Act was passed in 1896 (Chapter 313) granting further power to cities and towns to limit the heights of buildings on parkways and boulevards, the Act reserving to owners the right to recover damages by reason of such restrictions.

Following the precedent thus set by this general legislation, the Legislature in 1898 passed a Special Act (Chapter 452) limiting the height of buildings on or near Copley Square in Boston, this Act also reserving to owners the right to recover damages by reason of the restrictions imposed. The Special Act of 1898 was contested and its constitutionality upheld by a decision of the Supreme Court in 1899 (Attorney General *vs.* Williams, 174 Mass. 476).

The Court held the Act to be constitutional because:

"The statute gives rights in the nature of an easement over lands facing Copley Square, which easement is annexed to the Square for the benefit of the public, for whose use and enjoyment Copley Square was laid out; and * * * these rights are similar in their nature to rights in highways, in great ponds, and in the navigable waters of the Commonwealth. * * * In all respects the statute is in accordance with the laws regulating the taking of property by right of eminent domain, if the Legislature properly could determine that the preservation or improvement of the park in this particular was for a public use."

In regard to limitation of buildings around Copley Square as relating to public use, the Court was of the opinion that:

"If the Legislature, for the benefit of the public, was seeking to promote the beauty and attractiveness of a public park in the capital of the Commonwealth, and to prevent unreasonable encroachments upon the light and air which it had previously received, we cannot say that the law-making power might not determine that this was a matter of such public interest as to call for an expenditure of public money, and to justify the taking of private property. * * * It hardly would be contended that the same reasons which justify the taking of land for a public park do not also justify the expenditure of money to make the park attractive and educational to those whose tastes are being formed and whose love of beauty is being cultivated."

Conclusion

Massachusetts, in 1904, was the first State to apply the principle of excess condemnation to the taking of lot remnants in connection with street improvements. Massachusetts, in 1911, was the first to provide through amendment to its Constitution, authority for the broader exercise of excess condemnation in connection with street improvements so as to secure "suitable building lots on both sides of such highway or street". In Massachusetts, the principle has been established that parkways and boulevards may be protected by the taking of easements in abutting property, restricting the construction of buildings thereon. Thus, Massachusetts has laid the foundation for the protection and the adequate development of street improvements through the reasonable, although limited, exercise of excess condemnation. Except for the taking of easements and the condemnation of fee in lot remnants, these powers have not yet been utilized, however, and no project for street improvements involving the broad application of excess condemnation has been put into execution in Massachusetts.

In two other directions, which strictly speaking may not be interpreted as excess condemnation, but which are close to the border line, Massachusetts has carried out public improvements by State agency. These are the reclamation and development of tidal flats, work which has been going on for seventy-five years; and the provision of homes for its citizens, made possible by constitutional amendment, and for which thus far one small project has been built.

MULTIPLE-ARCH DAM AT GEM LAKE ON RUSH CREEK, CALIFORNIA

Discussion*

By J. Y. Jewett, Assoc. M. Am. Soc. C. E.

J. Y. Jewett,† Assoc. M. Am. Soc. C. E. (by letter).‡—In the early summer of 1923, the writer was called on to examine the structure referred to in this paper, and make a report on certain features of concrete construction related thereto. The repairs, involving use of the "Ironite" process, were at that time under way. Later, as mentioned in the paper, he was asked to make some tests of water-proofing materials, with a view to finding a suitable method of treatment for water-proofing the up-stream face of the dam, a brief description of which tests may be of interest.

The first series of these tests was confined principally to bituminous materials. Engineers of the Company felt that the solution of the problem might lie in the use of some substance having the flexible and elastic properties of this type of material, in contrast to rigid materials of the Portland cement mortar type which readily crack and allow the entrance of water under the severe temperature changes of that locality. The main points to be determined were: Durability under the prevailing climatic conditions; resistance to water pressure; bonding with the concrete surface; and penetration into the pores of the concrete under pressure.

Manufacturers of water-proofing materials of this type were asked to furnish samples of products which they could recommend as being especially resistant to the range of temperature involved. The samples received were classed in three groups, as follows:

(*a*) Solid material, to be melted and applied hot.

(*b*) Liquid material, to be applied with a brush as a paint.

(*c*) Material in mastic or putty form, to be applied with a trowel.

For practically all the samples submitted, the use of a light asphaltic paint as a primer coat was prescribed, and this was generally furnished with the sample.

In tests for resistance to water pressure, a pipe apparatus connected with the city system was used, on which a pressure of 55 to 60 lb. per sq. in. was available. The test specimens were 2 in. thick and 5 in. in diameter, coated on top with the material to be tested. These specimens were bolted between plates, with gaskets to make a tight joint, leaving an area, 3 in. in diameter, in the center exposed to water pressure, with provision for catching and measuring any water coming through. To determine absorption under pressure

* This discussion (of the paper by Fred O. Dolson and Walter L. Huber, Members, Am. Soc. C. E., to be presented at the meeting of October 7, 1925, and published in this number of *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Testing Engr., City of San Diego, San Diego, Calif.

‡ Received by the Secretary, June 13, 1925.

and penetration into the pores of the concrete, specimens 6 in. in diameter and 4 in. high, entirely coated with the material to be tested, were immersed in a closed tank under the same pressure. The effect of variation of temperature was observed by heating to 110° Fahr., and by exposure in cold storage at slightly below 0° Fahr.

All the materials except a light paint in Group (*b*) formed a coating which was water-tight under the pipe-pressure test. The tank apparatus did not prove to be adaptable to the purpose for which it was used, on account of the unbalanced condition caused by difference between air in the pores of the concrete at atmospheric pressure and the higher pressure maintained in the tank. This produced rupture of the enclosing membrane, and caused it to puff out in small bubbles and pull away from the surface of the concrete. In no case was there any penetration into the pores of the concrete.

Under the conditions prevailing at this dam, application of the materials of Group (*a*) would be difficult. The hot melted material cools very quickly on striking the cold surface of the concrete, and tends to stick to the tool with which it is being applied, and to pull away from the surface in a sheet. The materials of Group (*c*), although possessing spongy, elastic properties, were in general too soft and easily abraded to form a satisfactory coating, and some of them became softer under water than in the air. Under an increase of temperature up to 110° Fahr. a few of the samples remained unchanged, but the general effect was to soften the material, although not sufficiently to cause flow or change of shape. The specimens placed in cold storage were recently removed after one year's continuous exposure, and were practically unchanged in condition, the coatings showing no sign of cracking or peeling. To show the effect of varying conditions of temperature and moisture as existing at the structure, field exposure tests would be needed.

The conclusion is that if material of this type is used, it should be in paint form, of the class included in Group (*b*). Some of the paints of this group are of heavy consistency, about as thick as can be readily handled with a brush, and dry with a hard, smooth, tenacious coating. Among the several samples having these characteristics, a difference in hardness and tenacity under water immersion was noticed. Trial of a combination coating of such a paint over the mastics of Group (*c*) indicated that, while retaining the advantages of the former, some benefit would also accrue from the elastic properties of the latter.

A second series of tests was made on samples of general water-proofing compounds of the type intended to be applied as surface coatings in paint form. These materials, although showing properties which might be useful in certain classes of work, did not develop results under water pressure that would recommend them for the purpose in question. This series included several brands of the so-called water-proof, plastic, Portland cements now on the market. The plasticity of these products is doubtless of benefit as an aid to workability, in some classes of work, but they did not show water-proofing qualities in these tests equal to those of straight Portland cement. A marked feature of these cements was their low strength in compressive tests of concrete, as compared with straight Portland cement.

To the writer, in view of the results outlined, the problem seems to narrow down to a choice between two methods of treatment:

First.—The use of bituminous paint of the type described. This, although an unusual form of treatment for a structure of this kind, gives indications of furnishing an effective water-proof coating at a relatively low cost. If, however, it should need renewal from time to time, as may be the case, the cost would be a continuing item.

Second.—The use of a Portland cement mortar or concrete coating of high density, reinforced against temperature effects. Although the original cement-gun coating did not prove to be satisfactory, the usual product of this process is well known as a material of high density, and it should be possible to apply such a coating in this case so as to obtain advantage of this feature; or the procedure could be carried a step further, taking the form of a coating of reinforced concrete, similar to that of which the walls of the concrete ships are built. Without entering into discussion of this type of ship in general, it may be noted that they have shown the possibility of concrete construction in thin walls, of high density. As an alternative to the use of Portland cement in this proposed mortar or concrete coating, the use of a quick-hardening, high-strength cement of the newly developed "alumina" type deserves consideration.

With reference to the discussion by the authors of the concrete in the original structure, it was evidently considered that the work was of a good grade of concrete construction, and that the gunite facing could be relied upon for water-tightness. Tests of the concrete, however, as reported* by L. R. Jorgensen, M. Am. Soc. C. E., and as obtained on the specimens taken from the structure in 1924, show a low compressive strength for the mix used. One feature contributing to this, judging from the writer's observation, was the presence in the rock used as aggregate—which was obtained from material near the site of the work—of thin, flat, smooth-surfaced pieces, which would have somewhat the same effect in concrete as mica in a sand mortar. The authors, in describing the new construction, refer to the selection of a more suitable rock at some distance from the dam site.

Experience with this structure indicates that concrete to be used under similar conditions should be of what may be termed a super-excellent grade. This raises the question of the adequacy of the ordinary contract system (with its natural emphasis on speed of operation and profit for the contractor) to meet such a condition. From the point of view of a testing engineer, who, from his study of materials, perhaps realizes more fully than others the possibilities inherent in concrete as a construction material, a more effective method would be to place such work directly in charge of an engineer familiar with the problem and give him a free hand to get the results desired. This is subject, of course, to the limitation that there be no waste or needless expense; but it does leave such a man free to devote his energies to obtaining these results, rather than to expend them in more or less futile inspection efforts under the contract system.

* "Multiple-Arch Dams on Rush Creek, California", *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 850.

SECONDARY STRESSES IN BRIDGES

Discussion*

By Messrs. R. McC. Beanfield and Cecil Vivian von Abo.†

R. McC. Beanfield,‡ Assoc. M. Am. Soc. C. E. (by letter).§—The author's comparative analyses of various methods and conclusions involving a graphical and analytical solution for computing secondary stresses constitute valuable additional data on a technical subject lacking in comprehensive information of a practical nature.

Most of the accepted standard methods used to compute secondary stresses involve theoretical assumptions that do not correspond to conditions in actual practice. A few of these erroneous assumptions are as follows:

1.—The use of a constant moment of inertia throughout the theoretical length of the member, whereas in most instances the moment of inertia varies greatly, particularly at the end gusset connections, which in some cases is further aggravated by the use of clip angles and filler plates.

2.—The assumption that the length of the member is the theoretical length between intersection points. The theoretical length of the member as taken between axial intersection points may become considerably shortened by the use of large gusset-plates or other end conditions. If values ranging between the theoretical length and the free length (edge of gusset-plates) are substituted in the formula, a considerable variation in results is obtained.

3.—The restraining moment at the joint: In riveted connections considerable distortion may take place by reason of the elastic deformation of gusset-plates, the initial slip and distortion of rivets, and the frictional resistance between plates, all of which make the action of the joint under load rather uncertain and produce a complex condition of stresses, wherein it is practically impossible to determine accurately the restraining moment of the joint.‖

4.—The unknown frictional resistance of pin joints: Secondary stresses for pin connections are calculated in practically the same manner as for riveted connections. If the moment at the pin joint caused by the distortion of the truss exceeds the frictional resisting moment of the pin, then intersecting members may be assumed to rotate and to offer no resistance to moment. In other words, the secondary stresses in a pin joint are measured by the stresses caused by the frictional resistance of the pin. Thus, the

* Discussion on the paper by Cecil Vivian von Abo, Jun. Am. Soc. C. E., continued from August, 1925, *Proceedings*.

† Author's closure.

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§ Received by the Secretary, June 25, 1925.

‖ "Nickel Steel Riveted Joints", *Bulletin No. 49*, Univ. of Illinois; *Schweizerische Bauzeitung*, September 15, 1923.

accuracy of secondary stresses for pin-connected joints depends largely on the correctness of the coefficient of friction between the pin and its bearing. There are very meager data covering the frictional resistance of pins, therefore computed secondary stresses for pin-connected joints are based on highly theoretical assumptions which may give results varying greatly from actual conditions.

Recently, the writer had occasion to investigate the secondary stresses in a 168-ft. main balcony truss weighing approximately 250 tons, for the Shrine Auditorium at Los Angeles, Calif. A careful investigation of the secondary stresses was made because it was evident that the large, rigid, gusset connections would involve considerable restraining moments which were further complicated by reason of pin connections; some of the members had pin connections at one end and riveted connections at the other. The value of the frictional resistance between the nickel steel pins and the ordinary structural steel bearing was an unknown factor and had to be assumed.

Prior to making secondary stress computations for this truss, considerable study was given to the various accepted methods. An adaptation from Mohr's semi-graphical analytical system, as described by Mr. G. A. Maney,* was utilized and gave a clear and concise analysis, materially shorter than any of the methods suggested by the author.

The writer found some helpful data in a paper by the late F. C. Kunz, M. Am. Soc. C. E., entitled "Secondary Stresses".† Those interested in secondary stresses should read this paper carefully.

The secondary stresses in some of the members for the truss in the Shrine Auditorium were found to be too great and had to be reduced. All the secondary stresses were practically eliminated by erection methods that involved introducing initial stresses in the members by lengthening or shortening them as the deformation demanded.

In computing the secondary stresses of a number of large structural steel trusses in buildings the writer has found that the so-called approximate methods of solution, as proposed by Mohr and later suggested by Maney, gives sufficiently accurate results for practical purposes and involved considerably less labor than the method proposed by the author.

In many specifications, and in some building codes, secondary stresses are not clearly defined. Some authorities allow an increase in secondary stresses ranging from 10 to 33⅓ per cent. In other words, in complicated structures in which the stresses are difficult to analyze, relatively higher unit stresses are permitted, whereas in simple frame structures which have practically no secondary stresses, a much smaller unit stress is permitted.

Little data are available relative to secondary stresses in reinforced concrete frames, particularly long span trusses. It would be interesting to know what consideration was given to the secondary stresses in a number of long-span reinforced concrete trusses that have been erected.

* "Secondary Stresses and Other Problems in Rigid Frames," *Bulletin No. 1*, Univ. of Minnesota.

† *Engineering News-Record*, October 5, 1911.

The writer much prefers the reduction or elimination of secondary stresses in structural steel trusses by erection methods rather than by altering the design of the structure. The elimination of secondary stresses by proper erection methods does not involve complicated or expensive construction operations. As a matter of fact, the introduction of initial stresses by shortening or lengthening the members as the deformation demands and forcing them into position by jacks, or otherwise, proved to be a simple and comparatively inexpensive operation for the large balcony truss in the Shrine Auditorium in Los Angeles.

Secondary stresses should be investigated for all structural frames involving rigid or restrained joints, for even if the results may be approximate, they indicate the seriousness of neglecting them. The most efficient and economical method for reducing or eliminating secondary stresses requires considerable ingenuity on the part of the designing engineer who must be experienced and able to visualize as well as theorize.

CECIL VIVIAN VON ABO,* JUN. AM. SOC. C. E. (by letter).†—The writer is grateful to the Society for publishing his paper and to the many members who have contributed such useful information and criticism. The original idea was to investigate Dr. Mao's methods, but when the writer found them too cumbersome, he was advised by Professor Jacoby to make a thorough comparison of all existing methods. This in itself entailed a tremendous amount of work, and, therefore, he decided to confine his attention to what may be termed the more exact methods.

It was with great reluctance that, through lack of time, the writer had to forego the investigation of column action in compression members for publication in the paper, but the latter part of this discussion contains some interesting results.

Mr. Ammann‡ raised the point that, in order to take the gusset-plates into account, each bending moment calculated on the assumption of constant moments of inertia throughout the theoretical lengths of the members should be multiplied by the ratio of its theoretical length to the clear length between gussets. On the other hand, Mr. Steinman§ allows for an increasing moment of inertia toward the end of each member and modifies the fundamental equations. The writer is inclined to favor Mr. Steinman's method, as it seems to be the more logical.

The writer is indebted to Professor Cain‖ for pointing out an error in the paragraph following Equation (69).¶ Fixing attention on the second part of Equation (69), that is, $d\,v_4 = v_{4\,\psi} + u_{4\,\phi}$ and on Fig. 21,** $d\,v_4$ equals the sum of the counterclockwise moments of the ϕ's about u' plus the sum of the counterclockwise moments of the ψ's about v'. (Counterclockwise is negative in Mohr's

* Care, Chf. Civ. Engr., South African Rivers and Harbors, Johannesburg, Transvaal, South Africa.

† Received by the Secretary, July 16, 1925.

‡ *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1600.

§ *Loc. cit.*, February, 1925, Papers and Discussions, p. 297.

‖ *Loc. cit.*, December, 1924, Papers and Discussions, p. 1601.

¶ *Loc. cit.*, September, 1924, Papers and Discussions, p. 1001.

** *Loc. cit.*, p. 999.

"elastic weight" method). Not only are the axes, u, v, u', and v', turned clockwise through 90°, but also the ϕ's and the ψ's. This gives Professor Cain's Fig. 93.* Only the quantities, $(u_4 - u_1)$, $(v_4 - v_1)$, etc., remain unchanged. The first part of Equation (69), that is, $d\,u_4 = u_{4\,\psi} - v_{4\,\phi}$, interpreted by means of Fig. 93, then becomes:

$d\,u_4$ = the sum of the clockwise moments of the new ψ's about the new v'-axis — the sum of the counterclockwise moments of the new ϕ's about the new u'-axis;

that is,

$d\,u_4$ = — (the sum of the counterclockwise moments of the new ϕ's about the new u' + the sum of the counterclockwise moments of the new ψ's about the new v').

This change in sign would be troublesome if open chains were to be considered, but as the theory is extended to closed chains for which both these expressions then become zero (see Equation (72†)), the two parts of the equation mean one and the same thing. Any point on the closed chain is chosen as origin and any direction as that of the u-axis, which then fixes the direction of the v-axis as being at 90° clockwise to the u-axis. Equation (72), therefore, shows that the sum of the moments of the ϕ's (regarded as acting parallel to the u-axis) about the u-axis plus the sum of the moments of the ψ's (regarded as acting parallel to the v-axis) about the v-axis, is zero.

Professor Fuller‡ calls attention to the variations of stress across a section of a member recorded by him from observations in the field. It seems desirable that a theoretical investigation should be undertaken to determine all the principal variations of stress across sections of members of an existing bridge, due to a stationary locomotive, to be followed by a practical study when the more accurate extensometers could be used. If theory and practice agree, it would go far toward proving the validity of initial assumptions. Impact investigations could then very well follow, in order to get an idea of the factor for secondary and column-action stresses. The work would be stupendous, but if the outcome was a justification for the general acceptance of higher allowable unit stresses, the saving of metal in the bridges of the future would be considerable.

Messrs. Turner§ and Godfrey‖ both question the definition of "secondary stress". They have also raised some very difficult points which cannot be solved with absolute certainty mathematically. The end bending moments of a particular member are known fairly accurately and the writer demonstrates the importance to be attached to them.

It was very gratifying to note the research contributed by Dean Mackay.¶ It is hoped that the following investigation will account for some of the dis-

* *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1602.

† *Loc. cit.*, September, 1924, Papers and Discussions, p. 1002.

‡ *Loc. cit.*, December, 1924, Papers and Discussions, p. 1603.

§ *Loc. cit.*, January, 1925, Papers and Discussions, p. 130.

‖ *Loc. cit.*, April, 1925, Papers and Discussions, p. 644.

¶ *Loc. cit.*, February, 1925, Papers and Discussions, p. 289.

crepancies that he found in the field. It is shown, for example, that the variation in the bending moments along the length of a member is not linear. In tensile members, the bending moment curve will differ but little from a straight line, whereas in compression members it will differ widely. This would be still more marked if the unit compressive stress were increased.

Professor Beggs* is to be congratulated for his laboratory studies. It would save much calculation if one could obtain a close estimate of the secondary stresses in the manner he suggests.

Mr. Steinman is especially urged not to delay the publication of his simplified methods, for which he will have the gratitude of all bridge engineers. These methods will probably be the direct cause of a general adoption of higher unit stresses.

Professor Shedd† has provided a lucid explanation of the present status of secondary stresses, and Professor Cross‡ has given a modification of Mohr's semi-graphical method, which enhances its value. It is quite evident that a student of secondary stresses can fix his attention on the method of either Manderla or Mohr and know that he cannot do better. The writer found little to choose between them; in the light of the discussions, he would be quite prepared to give Mohr's method the premier position.

Professor Wilson§ has a remarkably fine method of solving the problem of cross-frames. It shows how small the range of individual research is with regard to the reading of all the literature on a particular subject. It is, therefore, requested that the Society amend the Bibliography‖ given by the writer by including such references as interested members of the Society can supply.

Many discussors have referred to the subject of column action and the following investigation may be of interest.

Column Action

Most of the work on the theory of column action involves complicated mathematics which makes the subject difficult to comprehend. The approximate theories are better known. They, however, are based on one of two assumptions: (1) That the ends of a column are entirely free to turn; and (2) that the ends are rigidly held. Neither assumption is true in practice. It would be interesting to consider the problem which accounts for some definite end bending moments.

Section A

Consider a member which is subjected to an axial thrust, two end bending moments, and a uniform load over its whole length. Assume the member to be straight initially and not to be bent much at any time, so that the radius of curvature at a point will be closely given by $R = \pm \frac{1}{\left(\frac{d^2 y}{d x^2}\right)}$. Let the

* *Proceedings*, Am. Soc. C. E., February, 1925, Papers and Discussions, p. 295.

† *Loc. cit.*, p. 298.

‡ *Loc. cit.*, March, 1925, Papers and Discussions, p. 406.

§ *Loc. cit.*, p. 411.

‖ *Loc. cit.*, September, 1924, Papers and Discussions, p. 1115.

modulus of elasticity and the moment of inertia be denoted by E and I, and regarded as constant. Fig. 110 shows the left-hand bending moment, M_1, acting clockwise and the right-hand bending moment, M_2, acting counterclockwise.

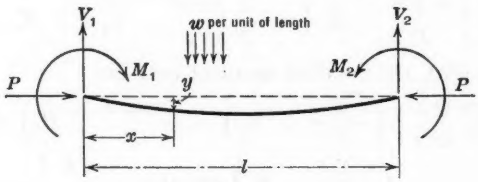

FIG. 110.

The two end shears are given by:

$$\left.\begin{aligned} V_1 &= \frac{w\,l}{2} - \frac{M_1 - M_2}{l} \\ V_2 &= \frac{w\,l}{2} + \frac{M_1 - M_2}{l} \end{aligned}\right\} \quad \ldots\ldots\ldots\ldots\ldots\ldots (122)$$

The bending moment at the point, x, is:

$$M_x = M_1 + P\,y + V_1\,x - \frac{w\,x^2}{2} \quad \ldots\ldots\ldots\ldots\ldots (123)$$

in which, y is the downward deflection of the member at x. Therefore,

$$E\,I\,\frac{d^2\,y}{d\,x^2} = -P\,y + \frac{w\,x^2}{2} - V_1\,x - M_1$$

that is,

$$E\,I\,\frac{d^2\,y}{d\,x^2} + P\,y = \frac{w\,x^2}{2} - V_1\,x - M_1 \quad \ldots\ldots\ldots\ldots (124)$$

The general solution of the differential equation is:

$$y = A\sin Q\,x + B\cos Q\,x + \frac{w\,x^2}{2\,P} - \frac{V_1\,x}{P} - \frac{M_1}{P} - \frac{w\,E\,I}{P^2} \ldots (125)$$

in which,

$$Q = \sqrt{\frac{P}{E\,I}}.$$

Now $y = 0$ when $x = 0$ and also when $x = l$. Therefore

$$B = \frac{w\,E\,I}{P^2} + \frac{M_1}{P}$$

and,

$$A = \frac{1}{P\sin Q\,l}\left\{\left(-\frac{w\,l^2}{2} + V_1\,l + M_1 + \frac{w\,E\,I}{P}\right) - \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,l\right\}$$

$$= \frac{1}{P\sin Q\,l}\left\{\left(\frac{w\,E\,I}{P} + M_2\right) - \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,l\right\}$$

using Equation (122). Hence, the solution is:

$$y = \frac{\sin Q\,x}{P \sin Q\,l}\left\{\left(\frac{w\,E\,I}{P} + M_2\right) - \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,l\right\}$$
$$+ \frac{\cos Q\,x}{P}\left\{\frac{w\,E\,I}{P} + M_1\right\} + \frac{1}{P}\left(\frac{w\,x^2}{2} - V_1\,x - M_1 - \frac{w\,E\,I}{P}\right) \ldots (126)$$

Using Equation (123), the bending moment at x is:

$$M_x = \frac{\sin Q\,x}{\sin Q\,l}\left\{\left(\frac{w\,E\,I}{P} + M_2\right) - \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,l\right\}$$
$$+ \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,x - \frac{w\,E\,I}{P} \ldots\ldots\ldots\ldots (127)$$

The maxima and minima points are given by:

$$\tan Q\,x = \frac{\left\{\left(\frac{w\,E\,I}{P} + M_2\right) - \left(\frac{w\,E\,I}{P} + M_1\right)\cos Q\,l\right\}}{\left(\frac{w\,E\,I}{P} + M_1\right)\sin Q\,l} \ldots (128)$$

Consider a few typical cases, as follows:

Case 1.—Consider a horizontal strut of uniform cross-section, under a compression of P lb. and assumed to have zero end moments. This is similar to Euler's assumptions of freely hinged ends. Equation (122) becomes,

$$V_1 = V_2 = \frac{w\,l^2}{2}$$

and Equation (128) becomes,

$$\tan Q\,x = \frac{1 - \cos Q\,l}{\sin Q\,l} = \tan\frac{Q\,l}{2} \ldots\ldots\ldots\ldots (129)$$

therefore,

$$Q\,x = \frac{Q\,l}{2} \pm n\,\pi,$$

in which n may be any integer. Therefore,

$$x = \frac{l}{2} \pm \frac{n\,\pi}{Q} \ldots\ldots\ldots\ldots (130)$$

provided $0 < x < l$.

Equation (127) becomes,

$$M_x = \frac{w\,E\,I}{P}\left(\tan\frac{Q\,l}{2} \times \sin Q\,x + \cos Q\,x - 1\right) \ldots\ldots (131)$$

As an example, take a 3 by 3 in. by ½-in. angle-iron, with an area of cross-section of 2.70 sq. in., a least moment of inertia of 0.90 in.⁴, and a length of 60 in., giving the ratio of l to r of somewhat more than 100. Let the strut be in the position of least strength, as shown in Fig. 111. If $Q\,l = \pi$, Equation (130) gives,

$$x = \frac{l}{2} \pm n\,l$$

and, therefore, n must be zero. For all values of Q making $Q\,l < \pi$, n will be found to be zero because Equation (130) will give,

$$x = \frac{l}{2} \pm \frac{n\,\pi\,l}{(\pi - \text{something})}$$

and x must lie between 0 and l. Hence, there is only one section to consider, namely the central one. $Q\ l = \pi$ was chosen both because it gives Euler's critical load and also because it is the value that makes M_x of Equation (131) infinite.

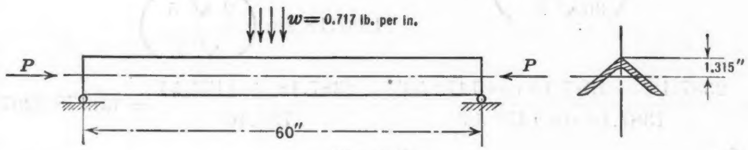

FIG. 111.

This critical load is,

$$\frac{\pi^2\ E\ I}{l^2} = \frac{\pi^2 \times 30 \times 10^6 \times 0.90}{60 \times 60} = 74\ 022 \text{ lb.}$$

taking $E = 30 \times 10^6$ lb. per sq. in.

Study the effect of various loads less than the critical: Starting with 50 000 lb., Equation (131) gives:

$$M_{\frac{l}{2}} = \frac{w\ E\ I}{P}\left(\tan\frac{Q\ l}{2} \cdot \sin\frac{Q\ l}{2} + \cos\frac{Q\ l}{2} - 1\right) = \frac{w\ E\ I}{P}\left(\sec\frac{Q\ l}{2} - 1\right)$$

$$Q = \sqrt{\frac{5 \times 10^4}{27 \times 10^6}};\quad \frac{Q\ l}{2} = \sqrt{\frac{5}{3}} \text{ radians} = 73^\circ\ 58';$$

$$\frac{w\ E\ I}{P} = \frac{717 \times 27 \times 10^6}{5 \times 10^4} = 387.18.$$

Therefore,

$$M_{\frac{l}{2}} = 387.18\ (3.6206 - 1) = 1\ 014.65 \text{ in-lb.}$$

Similarly, the results for loads of 55 000 lb., etc., are given in Table 39 and Fig. 112.

TABLE 39.

Total load, in pounds.	Primary stress, in pounds per square inch.	Maximum bending moment (at center), in inch-pounds.	Secondary stress (top fiber), in pounds per square inch.	Total stress (top fiber), in pounds per square inch.
50 000	18 520	1 015	1 480	20 000
55 000	20 370	1 285	1 880	22 250
60 000	22 220	1 747	2 550	24 470
65 000	24 070	2 719	3 970	28 040
70 000	25 930	6 106	8 920	34 850
72 500	26 850	16 125	23 560	50 410
74 022	27 420	......	......	......

Case 2.—Suppose two end bending moments are introduced into the foregoing problem, both assisting the weight of the beam. Take $M_1 = +\ 1\ 000$ in-

lb. and $M_2 = + 2\,000$ in-lb. Assuming a load of 50 000 lb., Equation (128) gives:

$$\tan\left(\frac{\sqrt{5}}{30\sqrt{3}}x\right)^c = \frac{2387.18 - 1387.18\cos\left(\frac{2\sqrt{5}}{\sqrt{3}}\right)}{1387.18\sin\left(\frac{2\sqrt{5}}{\sqrt{3}}\right)}$$

$$= \frac{2387.18 - 1387.18\cos 147°\,56'}{1387.18\sin 147°\,56'} = \frac{2387.18 + 1175.54}{736.46} = \tan 78.3207°$$

Therefore,

$$x = \frac{78.3207\,\pi}{180} \times \frac{30\sqrt{3}}{\sqrt{5}} = 31.765 \text{ in.}$$

and,

$$M_{max.} = M_{31.765\text{ in.}} = \frac{2387.18 - 1387.18\cos 147°\,56'}{\sin 147°\,56'} \times \sin 78°\,19'$$

$$+ 1387.18\cos 78°\,19' - 387.18 = 6571.6 + 280.9 - 387.18 = 6465.3 \text{ in-lb.}$$

Similarly, the results for loads of 55 000 lb., etc., are given in Table 40 and Fig. 113.

TABLE 40.

Total load, in pounds.	Primary stress, in pounds per square inch.	Maximum bending moments, in inch-pounds.	Distance from left-hand end, in inches.	Secondary stress (top fiber), in pounds per square inch.	Total stress (top fiber), in pounds per square inch.
50 000	18 520	6 465	31 765	9 450	27 970
55 000	20 370	8 276	31 317	12 090	32 460
60 000	22 220	11 382	30 920	16 630	38 850
65 000	24 070	17 930	30 562	26 200	50 270
70 000	25 930	40 790	30 239	59 600	85 530

Case 3.—Assume two end bending moments, both in opposition to the dead weight, namely, $M_1 = -1\,000$ in-lb. and $M_2 = -2\,000$ in-lb. Considering a load of 50 000 lb., Equation (128) gives,

$$\tan\left(\frac{\sqrt{5}}{30\sqrt{3}}x\right)^c = \frac{-1612.82 + 612.82\cos 147°\,56'}{-612.82\sin 147°\,56'} = \frac{2132.14}{325.25} = \tan 81.324°.$$

Therefore,

$$x = \frac{81.324\,\pi}{180} \times \frac{30\sqrt{3}}{\sqrt{5}} = 32.983 \text{ in.}$$

and,

$$M_{max.} = M_{32.983\text{ in.}} = -\frac{2132.14}{\sin 147°\,56'} \times \sin 81°\,19' - 612.82\cos 81°\,19' - 387.18$$

$$= -4449.7 \text{ in-lb.}$$

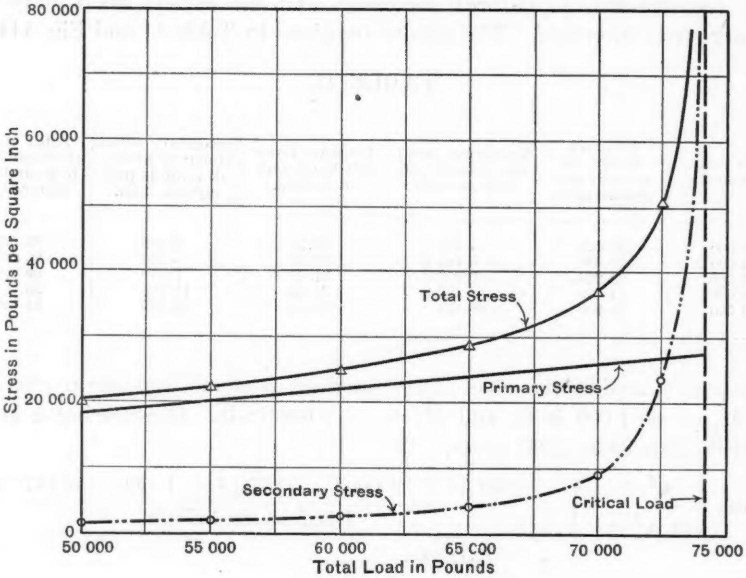

FIG. 112.

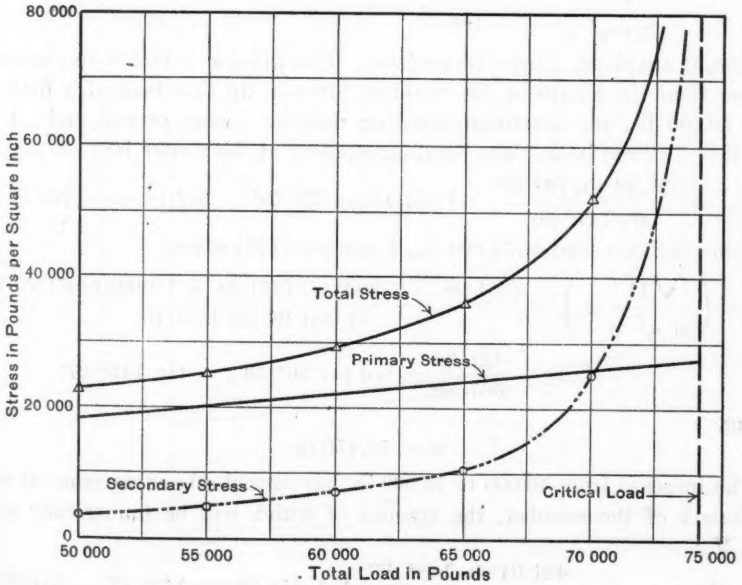

FIG. 113.

The negative bending moments mean that the top fibers have their primary stress (compressive in nature) decreased, and the bottom fibers have their primary stress increased. The results are given in Table 41 and Fig. 114.

TABLE 41.

Total load, in pounds.	Primary stress, in pounds per square inch.	Maximum bending moment, in inch-pounds.	Distance from left-hand end, in inches.	Secondary stress (bottom fiber), in pounds per square inch.	Total stress, (bottom fiber), in pounds per square inch.
50 000	18 520	— 4 449.7	32.98	3 985	22 505
55 000	20 370	— 5 715.6	32.33	5 120	25 490
60 000	22 220	— 7 893.4	31.05	7 070	29 290
65 000	24 070	—12 492	30.84	11 190	35 260
70 000	25 930	— 28 573	30.35	25 590	51 520

Case 4.—Consider the effect of end bending moments, opposite in character, say, $M_1 = + 1\,000$ in-lb. and $M_2 = - 2\,000$ in-lb. Considering a load of 50 000 lb., Equation (128) gives,

$$\tan\left(\frac{\sqrt{5}}{30\sqrt{3}}\,x\right) = \frac{(387.18 - 2\,000) - (387.18 + 1\,000)\cos 147° \, 56'}{1\,387.18 \sin 147° \, 56'}$$

$$= -\frac{437.28}{736.46}$$

giving,

$$\frac{\sqrt{5}\,x}{30\sqrt{3}} = -\frac{30.7}{180}\pi + \pi,$$

because, if anything, x must be positive. This gives $x = 60.553$ in., an amount greater than the length of the member. Hence, up to a load of a little more than 50 000 lb., the maximum bending moment occurs at one end. In this case it is — 2 000 in-lb. The bending moment at the center is:

$$M_{30} = -\frac{437.28 \sin 73° \, 58'}{\sin 147° \, 56'} + 1\,387.18 \cos 73° \, 58' - 387.18 = - 795.66 \text{ in-lb.}$$

Considering a load of 55 000 lb., Equation (128) gives:

$$\tan\left(\frac{\sqrt{11}}{30\sqrt{6}}\,x\right) = \frac{(351.98 - 2\,000) - (351.98 + 1\,000)\cos 155° \, 10'}{1\,351.98 \sin 155° \, 10'}$$

$$= -\frac{421.04}{567.80} = \tan(- 36° \, 33') = \tan 143° \, 27'$$

giving,

$$x = 55.470 \text{ in.}$$

The increase from 50 000 to 55 000 lb. has caused a bending moment within the length of the member, the amount of which will be numerically greater than M_2:

$$M_{55.470 \text{ in.}} = -\frac{421.04 \sin 143° \, 27'}{\sin 155° \, 10'} + 1\,351.98 \cos 143° \, 27' - 351.98$$

$$= - 2\,035.04 \text{ in-lb.}$$

Similarly, for 60 000 lb., etc., the results are given in Table 42 and Fig. 115.

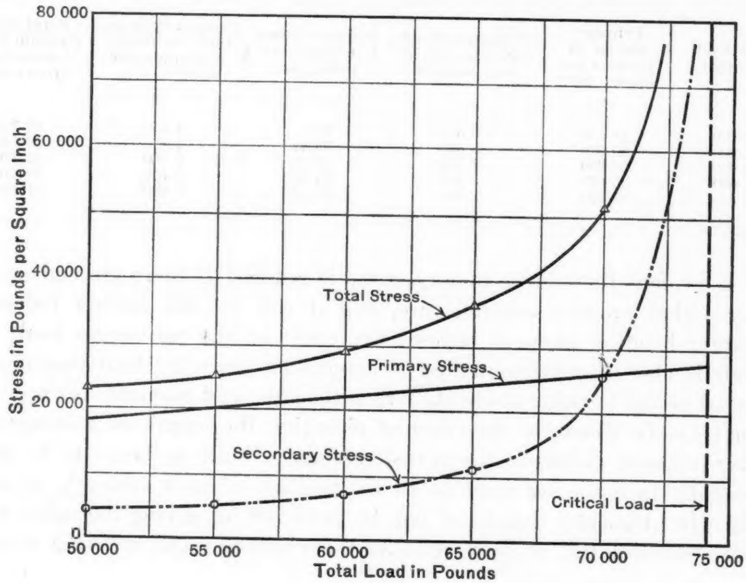

FIG. 114.

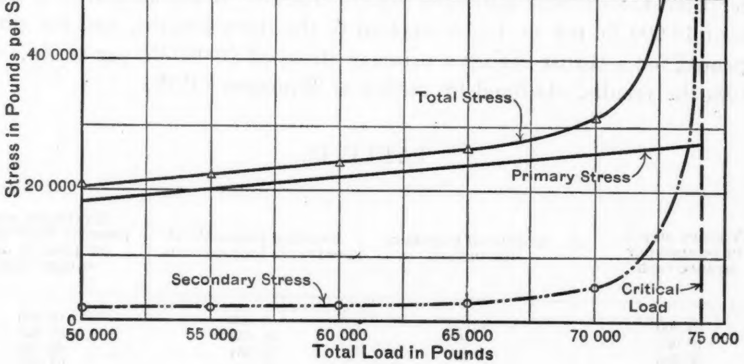

FIG. 115.

TABLE 42.

Total load, in pounds.	Primary stress, in pounds per square inch.	Maximum bending moment, in inch-pounds.	Distance from left-hand end, in inches.	Secondary stress (bottom fiber), in pounds per square inch.	Total stress (bottom fiber), in pounds per square inch.
50 000	18 520	−2 000	60	1 790	20 310
55 000	20 370	−2 035	55.47	1 820	22 190
60 000	22 220	−2 220	49.68	2 000	24 220
65 000	24 070	−2 840	42.93	2 540	26 610
70 000	25 930	−5 808	35.42	5 200	31 130

It is evident that the secondary stress is reduced if there are end bending moments that produce contraflexure, and it will be still further reduced if the larger bending moment opposes the effect of the transverse load. The analysis is also interesting in that it determines the axial load that must be subjected before bending moments larger than the end moments occur.

Case 5.—To illustrate the effect of changing the length of a compression member without changing the thrust, assume a built-up beam to be placed horizontally in a testing machine in the position of least strength, as shown on Fig. 116. Consider lengths of 120, 160, and 200 in., giving the ratios of l to r of 75, 100, and 120, respectively. As there will be some resisting moments

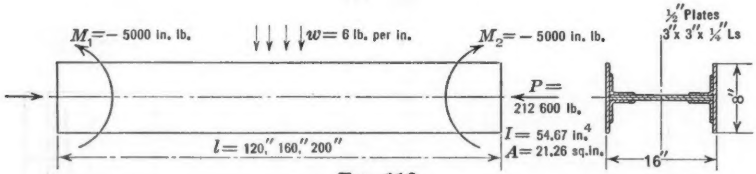

FIG. 116.

at the ends, assume them to be 5 000 in-lb. each. Up to the critical load, the maximum bending moment will occur at the center of the beam. Loads greater than the critical value are not considered. A thrust giving a primary stress of 10 000 lb. per sq. in., is applied to the three lengths, and the problem is repeated for a thrust giving a primary stress of 20 000 lb. per sq. in. Table 43 gives the results, obtained by means of Equation (127).

TABLE 43.

Primary stress, in pounds per square inch.	Length of member, in inches.	Bending moment, at center, in inch-pounds.	Maximum compressive fiber stress, in pounds per square inch.
10 000	120	6 508	10 465
10 000	160	21 052	11 540
10 000	200	52 264	13 825
20 000	120	26 364	21 930
20 000	160	77 764	25 690
20 000	200	Load greater than critical load.	

Section B.—Uniform Transverse Load Over Part of Span

Consider the problem of a compression member with an axial load, two end bending moments, and a uniform transverse load over part of the span, all acting in one plane, as shown in Fig. 117.

For $0 < x < a$,

$$M_x = M_1 + V_1 x + P y \quad \text{(132)}$$

for $a < x < (a + b)$,

$$M_x = M_1 + V_1 x - \frac{w (x - a)^2}{2} + P y \quad \text{(133)}$$

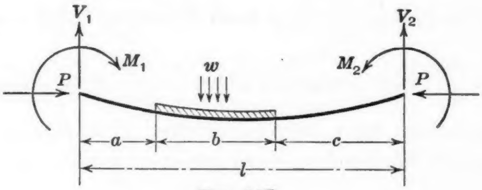

Fig. 117.

and for $(a + b) < x < l$,

$$M_x = M_1 + V_1 x - w b \left(x - a - \frac{b}{2}\right) + P y \quad \text{(134)}$$

$$\left.\begin{aligned} V_1 &= \frac{w b \left(c + \frac{b}{2}\right)}{l} - \frac{M_1 - M_2}{l} \\ V_2 &= \frac{w b \left(a + \frac{b}{2}\right)}{l} + \frac{M_1 - M_2}{l} \end{aligned}\right\} \quad \text{(135)}$$

The differential equations are:

For $0 < x < a$,

$$E I \frac{d^2 y}{d x^2} + P y = - V_1 x - M_1 \quad \text{(136)}$$

for $a < x (a + b)$,

$$E I \frac{d^2 y}{d x^2} + P y = \frac{w x^2}{2} - (V_1 + w a) x - \left(M_1 - \frac{w a^2}{2}\right) \quad \text{(137)}$$

and for $(a + b) < x < l$,

$$E I \frac{d^2 y}{d x^2} + P y = - (V_1 - w b) x - \left\{ M_1 + w b \left(a + \frac{b}{2}\right) \right\} \quad \text{(138)}$$

The general solutions are:

For $0 < x < a$,

$$y = A \sin Q x + B \cos Q x - \frac{V_1 x}{P} - \frac{M_1}{P} \quad \text{(139)}$$

for $a < x < (a + b)$,

$$y = C \sin Q x + D \cos Q x + \frac{w x^2}{2 P} - \frac{(V_1 + w a) x}{P} - \frac{\left(M_1 - \frac{w a^2}{2}\right)}{P} \quad \text{(140)}$$

and for $(a + b) < x < l$,

$$y = E \sin Q\,x + F \cos Q\,x - \frac{(V_1 - w\,b)\,x}{P} - \frac{\left\{ M_1 + w\,b \left(a + \frac{b}{2} \right) \right\}}{P} \qquad (141)$$

From the six conditions: — y from Equation (139) $= 0$, $x = 0$; y from Equation (139) $= y$ from Equation (140), $x = a$; $\frac{d\,y}{d\,x}$ from Equation (139), $= \frac{d\,y}{d\,x}$ from Equation (140), $x = a$; y from Equation (140) $= y$ from Equation (141), $x = a + b$; $\frac{d\,y}{d\,x}$ from Equation (140) $= \frac{d\,y}{d\,x}$ from Equation (141), $x = a + b$; y from Equation (141) $= 0$, $x = l$; the six constants are determined. The final equations for the bending moments are:

For $0 < x < a$,

$$M_x = \frac{\sin Q\,x}{\sin Q\,l} \left[\frac{w\,E\,I}{P} \left\{ \cos Q\,c - \cos Q\,(b + c) \right\} - M_1 \cos Q\,l + M_2 \right] + M \cos Q\,x \qquad (142)$$

for $a < x < a + b$,

$$M_x = \frac{\sin Q\,x}{\sin Q\,l} \left[\frac{w\,E\,I}{P} \left\{ \cos Q\,c - \cos Q\,a \,.\, \cos Q\,l \right\} - M_1 \cos Q\,l + M_2 \right] + \left[M_1 + \frac{w\,E\,I}{P} \cos Q\,a \right] \cos Q\,x - \frac{w\,E\,I}{P} \qquad (143)$$

and for $a + b < x < l$,

$$M_x = \frac{\sin Q\,x}{\sin Q\,l} \left[\frac{w\,E\,I}{P} \cos Q\,l \left\{ \cos Q\,(a + b) - \cos Q\,a \right\} - M_1 \cos Q\,l + M_2 \right] + \left[M_1 - \frac{w\,E\,I}{P} \left\{ \cos Q\,(a + b) - \cos Q\,a \right\} \right] \cos Q\,x \qquad (144)$$

Section C.—Concentrated Transverse Load

Mathematically, a concentrated load is considered to act at a point. If the problem of a concentrated transverse load is investigated, one arrives at a point of discontinuity for the bending moment just under the load. In practice, however, a concentrated load is always found to be spread over some definite distance. Equations (142), (143) and (144) are then used to determine the bending moments. A problem similar to the one given by Mr. Godfrey will be considered. The reference is to his Fig. 106 (*A*).* By virtue of the fact that the post was initially straight at the top, the forcing of a ball between the two parts of the post will create a large bending moment at the top, and this bending moment will act in opposition to the pressure of the ball. A certain bending moment will be found at the base as well, due to friction, etc. This, too, will oppose the action of the ball.

* *Proceedings*, Am. Soc. C. E., April, 1925, Papers and Discussions, p. 648.

Consider a built-up strut, held in a vertical position, with a load of 263 000 lb. on it and a horizontal pressure of 10 000 lb. at its center, acting in the direction of least strength of the strut and assumed to be acting over a space of 4 in. Fig. 118 shows the strut in a horizontal position.

Let,

$$I = 106.93 \text{ in.}^4;$$

$$\text{Area} = 26.3 \text{ sq. in.};$$

$$w = 2\,500 \text{ lb. per in.};$$

$$\frac{P}{E\,I} = Q^2 = \frac{263\,000}{30 \times 10^6 \times 106.93};$$

$$Q\,l = 62^\circ\ 15';$$

$$Q\,c = Q\,a = 30^\circ\ 5';$$

$$Q\,(a + b) = Q\,(b + c) = 32^\circ\ 10'$$

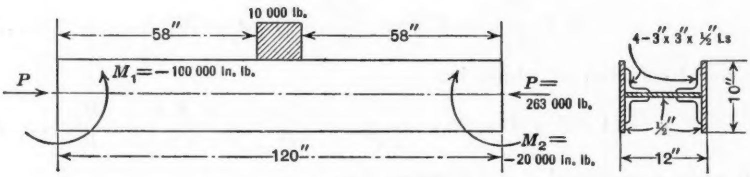

FIG. 118.

Substituting in Equations (142) and (144) and differentiating, no maxima points are found within Spaces a and c, but Equation (143) gives $Q\,x = 31^\circ$ $16'$ for the point of maximum bending moment. The center of the strut is given by $Q\,x = 31^\circ\ 7\tfrac{1}{2}'$, so the maximum bending moment occurs just beyond the center.

$$\frac{w\,E\,I}{P} = 3.049335 \times 10^7$$

$$M_{\text{max.}} = \frac{\sin 31^\circ\ 16'}{\sin 62^\circ\ 15'} \Big[3.049335 \times 10^7 \times \cos 30^\circ\ 5'\ (1 - \cos 62^\circ\ 15')$$

$$+ 10^5 \cos 62^\circ\ 15' - 2 \times 10^4 \Big] + \Big[3.049335 \times 10^7 \times \cos 30^\circ\ 5' - 10^5 \Big]$$

$$\cos 31^\circ\ 16' - 30\,493\,350 = 259\,700 \text{ in-lb.}$$

$$M_{60 \text{ in.}} = 259\,600 \text{ in-lb.}$$

$$M_{58 \text{ in.}} = 253\,050 \text{ in-lb.}$$

Upon solving the problem under the supposition of freely-hinged ends:

$$M_{\text{max.}} = M_{\text{center}} = 295\,900 \text{ in-lb.}$$

Section D.—Strut, Originally Straight, Subjected to an Eccentric Load

If P of Fig. 110 is off center, a positive distance, e, Equation (123) becomes:

$$M_x = M_1 + P\,(e + y) + V_1\,x - \frac{w\,x^2}{2},$$

resulting in changes of M_1 to $M_1 + Pe$ and M_2 to $M_2 + Pe$ in Equation (127). Hence, for eccentric loading,

$$M_x = \frac{\sin Q\,x}{\sin Q\,l}\left[\left(\frac{w\,E\,I}{P} + M_2 + P\,e\right) - \left(\frac{w\,E\,I}{P} + M_1 + P\,e\right)\cos Q\,l\right]$$
$$+ \left(\frac{w\,E\,I}{P} + M_1 + P\,e\right)\cos Q\,x - \frac{w\,E\,I}{P} \ldots\ldots\ldots\ldots (145)$$

Similar changes occur in the Equations (142), (143), and (144).

Section E.—Tension Members

Consider the case of a tension member subjected to transverse loading, that is, assume P of Fig. 110 to act in the opposite direction. Equation (123) becomes:

$$M_x = M_1 + V_1\,x - P\,y - \frac{w\,x^2}{2} \ldots\ldots\ldots\ldots (146)$$

Equation (124) becomes:

$$E\,I.\frac{d^2\,y}{d\,x^2} - P\,y = \frac{w\,x^2}{2} - V_1\,x - M_1 \ldots\ldots\ldots\ldots (147)$$

the general solution of which is:

$$y = A\,e^{Qx} + B\,e^{-Qx} - \frac{w\,x^2}{2\,P} + \frac{V_1\,x}{P} - \frac{w\,E\,I}{P^2} + \frac{M_1}{P} \ldots\ldots (148)$$

$y = 0$ when $x = 0$, and $x = l$ gives:

$$A = \frac{\left(\frac{w\,E\,I}{P^2} - \frac{M_2}{P}\right) - \left(\frac{w\,E\,I}{P^2} - \frac{M_1}{P}\right)e^{-Ql}}{(e^{Ql} - e^{-Ql})}$$

and,

$$B = \frac{\left(\frac{w\,E\,I}{P^2} - \frac{M_1}{P}\right)e^{Ql} - \left(\frac{w\,E\,I}{P^2} - \frac{M_2}{P}\right)}{(e^{Ql} - e^{-Ql})}$$

The equation for the bending moment at x is:

$$M_x = \frac{w\,E\,I}{P} - \frac{\left(\frac{w\,E\,I}{P} - M_2\right) - \left(\frac{w\,E\,I}{P} - M_1\right)e^{-Ql}}{(e^{Ql} - e^{-Ql})} \times e^{Qx}$$
$$- \frac{\left(\frac{w\,E\,I}{P} - M_1\right)e^{Ql} - \left(\frac{w\,E\,I}{P} - M_2\right)}{(e^{Ql} - e^{-Ql})} \times e^{-Qx} \ldots\ldots\ldots (149)$$

Section F.—Compression Members, Subjected to Pairs of End Bending Moments in Planes at Right Angles to Each Other.

This case is perhaps the most important. In considering the top chord of a bridge, end bending moments are found that act in a perpendicular plane and are due to the rigidity of the truss, and also those in the horizontal plane due to the rigidity of the lateral system. The resultant of the pair at one end is inclined at some angle to the vertical plane. If the resultant of the pair at the other end, the resultant of the uniform dead weight of the member, and the uniform wind load were to act in the same inclined plane, one

would perhaps be justified in applying the theory of unsymmetrical bending.* This supposition is hardly ever true. It seems, however, that if the forces and moments in the vertical and horizontal planes are studied separately and the resulting bending moments at any particular section combined, a result would be determined which should be close to, if not actually, the true value.

As an illustration, investigate the chord member, 2–4, of the bridge studied by the writer in his paper. From the results given on pages 1112 and 1113† the maximum thrust is found to be 510 500 lb. and the vertical end moments are — 104 680 and + 335 800 in-lb. for Ends (2) and (4), respectively. The uniform dead load is estimated to be 16 lb. per in. and the moment of inertia is 2385.4 in.^{4} The horizontal end moments are + 275 040 and + 355 880 in-lb. for Ends (2) and (4), respectively, while the wind load, assumed to be acting on the outside of the chord, is taken as 6 lb. per lin. in., which is rather a high estimate. The moment of inertia to be used in this problem is 2852.45 in.^{4} The length of the member is 300 in. and E is taken as 30×10^6 lb. per sq. in. (See Fig. 119.)

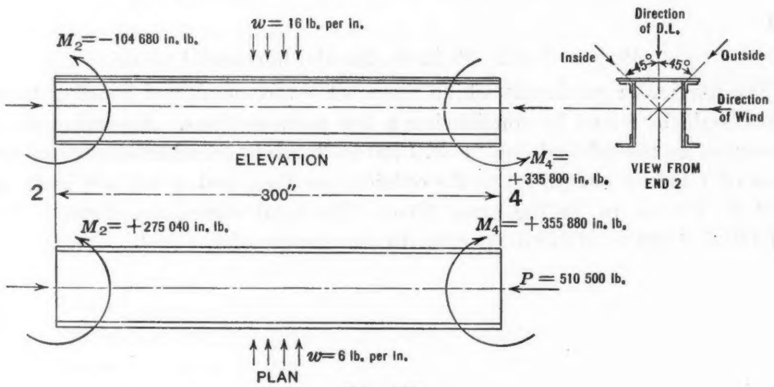

Fig. 119.

Consider the effect at the center: Equation (127) gives,

$M_c = + 318\,360$ in-lb. for the vertical plane.

and

$M_c = + 409\,380$ in-lb. for the horizontal plane.

The plus signs denote that the bending moments are increasing the compressive stresses in the top and the outside fibers, respectively.

By way of determining the effect in planes other than the vertical and horizontal, the two 45° planes were considered. For the plane inclined from the outside the combined end bending moments were + 120 460 and 489 090 in-lb. for Ends (2) and (4), respectively. The plane of bending is determined

* "Modern Framed Structures", Vol. III, Johnson, Bryan, and Turneaure.

† *Proceedings*, Am. Soc. C. E., September, 1924, Papers and Discussions.

by $\tan^{-1}\left(\frac{I_x}{I_y}\cot 45^\circ\right)$, that is, 39° 54¼′ to the horizontal, giving a moment of inertia of,

$$I_x \cos^2 39^\circ\ 54\tfrac{1}{4}' + I_y \sin^2 39^\circ\ 54\tfrac{1}{4}',$$

that is, 2 577.6 in.[4] The bending moment at the center was found to be $M_c = +$ 515 460 in-lb. Similarly, the bending moment at the center for the plane inclined from the inside was found to be $M_c = -$ 67 740 in-lb. The results, $M_c = +$ 318 360 and $M_c = +$ 409 380, for the vertical and horizontal planes combine to give $\frac{318\ 360 + 409\ 380}{\sqrt{2}}$ and $\frac{318\ 360 - 409\ 380}{\sqrt{2}}$, that is + 514 590 and — 64 360 in-lb., for the respective inclined planes. There are, therefore, differences in the results which seem to point to further research in the theory of unsymmetrical bending.

Assuming, however, that combinations of the results obtained from the vertical and horizontal planes give true values, consider the effect at the section, 200 in. from End (2). Equation (127) gives:

$$M_{200} = +\ 370\ 750 \text{ in-lb. for the vertical plane}$$

and

$$M_{200} = +\ 412\ 590 \text{ in-lb. for the horizontal plane}$$

The particular section at which the most severe combined bending moment exists could be found by considering a few more sections. Assuming the 200-in. section as the critical one, + 370 750 in-lb. gives an additional compressive stress of 1 450 lb. per sq. in. to the outside top fiber, and + 412 590 in-lb. gives 1 446 lb. per sq. in. to the same fiber. The total stress, therefore, is 11 782 + 1 450 + 1 446 = 14 678 lb. per sq. in. in compression.

FLOOD FLOW CHARACTERISTICS

Discussion*

By Robert E. Horton, M. Am. Soc. C. E.

Robert E. Horton,† M. Am. Soc. C. E. (by letter).‡—The author adopts tentatively the Myers formula in which flood discharge varies as the square root of the area. In most such formulas, larger exponents of A have been used, generally $\frac{3}{4}$, $\frac{4}{5}$ or $\frac{5}{6}$. The author does not give reasons for the use of so small an exponent of A; presumably he believes it better adapted to Western streams.

The average slope of the line of relation between Q and A could be determined by taking group means of all the data plotted on Plate VIII§ within the limits of selected increments of A and plotting the mean values of Q in terms of A. This the author has not done. The data on Plate VIII seem to show some justification for the reduction of the exponent of A below its customary value. A line through the center of gravity of the points having a slope corresponding, say, to $A^{0.8}$ would clearly fall below the majority of the points to the left and above a majority of those to the right of the central point. It is to be noted, however, that in nearly all cases where the exponent of Q is taken in terms of the $\frac{3}{4}$ to $\frac{5}{6}$ power of A, slope is also included as a factor in the formula. Large areas are generally flatter than small ones. The steeper the slope the greater the flood discharge, other things being equal. The flood discharges recorded on Plate VIII for small areas are, therefore, greater than they would be if the slope of these small areas was the same as that of the larger areas; hence, to fit the plotted points the line must be steeper or, in other words, a smaller exponent of A must be used, than if the slope of the drainage basins was considered separately. However, it seems certain that the data given by the author would serve to evaluate the law of slope in relation to flood discharge quite as readily as the law of area. If this was done, one less factor would be left to the uncertainties of human judgment. The author's formula apparently does not relate to basins of any particular slope, but is based, tacitly at least, on the presumption that each basin has the average slope of all basins of the same area, this average slope differing for each size of area and being steeper for small than for large areas. This assumption

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* Discussion on the paper by C. S. Jarvis, M. Am. Soc. C. E., continued from August, 1925, *Proceedings*.

† Cons. Hydr. Engr., Albany, N. Y.

‡ Received by the Secretary, July 1, 1925.

§ *Proceedings*, Am. Soc. C. E., December, 1924, Papers and Discussions, p. 1555.

is hardly a safe guide in computing flood discharge in specific cases. Two basins of equal size in the same region may differ widely in slope. Drainage basins on the Great Plains are generally much flatter than basins of equal area either in the East, or in the Western mountain regions.

The author compares the Myers flood formula with the Kutter-Chezy formula for flow in channels and pipes. In the Kutter-Chezy formula two important variables, R and S, are directly included and only the effect of roughness is covered by the judgment coefficient, n. In the Myers formula only one of the two main variables is directly provided for, namely, area. Rainfall and all the factors which affect the rate of run-off from a given area are thrown into a judgment coefficient.

The writer believes it is better in general to reduce the factors depending on judgment to a minimum. That this can be done in some degree, at least, in flood discharge calculation, is evidenced by the general success attending the application of the so-called "rational method" to storm-sewer design. At present, the laws connecting surface slope, permeability of soil, and other factors with flood run-off rates are only imperfectly known. Methods by which data may be obtained for the correlation of some of these factors with flood discharge have been developed by the writer and are presented herewith, some of them for the first time.

In addition to area, the principal physiographic factors which are correlated with the flood discharge of a drainage basin are:

(1) Length of basin, measured along the main stream channel.
(2) Drainage density or length of streams per square mile of area.
(3) Average slope of the stream channels.
(4) Average slope of the ground surface.
(5) Slope ratio or ratio of stream channel slope to land surface slope.
(6) Average length of overland flow.

Quantitative values of these factors can be determined without great difficulty if suitable topographic maps are available.

Let,

$A =$ area, in square miles.
$\Sigma l =$ total length of streams in the drainage basin, in miles.
$h =$ contour interval used, in feet, measured vertically.
$\Sigma c =$ total length of contours of vertical interval, h, on a given area.
$S_c =$ average slope of stream channel.
$S_g =$ average slope of land surface.
$d =$ drainage density $= \frac{\Sigma l}{A}$.
$\frac{S_c}{S_g} =$ slope ratio.
$w =$ average distance between contributory streams.
$z =$ average horizontal angle between the direction of stream flow and the direction of overland flow.
$l_g =$ average length of overland flow.

The obvious method of determining drainage density is to measure with an opisometer the total length of the streams shown on a topographic map and divide this result by the area. For large areas this is laborious, and a sufficiently accurate value can generally be obtained by the "sample area method", which consists in determining the drainage density on a few selected representative rectangular areas and taking the average.

The slope of streams can be determined by noting the elevation of the source and mouth of each contributory and measuring its length. If ΣE_1 is the sum of the elevation of stream sources and ΣE_2 is the sum of the elevation of their mouths, then the mean slope of the streams, in feet per mile, is:

$$S_c = \frac{\Sigma E_1 - \Sigma E_2}{\Sigma l}$$

This determination can also be made by counting the number of contours intersecting all streams. Calling this number Σc:

$$S_c = \frac{h \Sigma c}{\Sigma l}.$$

The latter method has an advantage in that the small streams, which largely determine the average slope, are not so likely to be omitted. This determination should generally be made for the whole basin, although the "sample area method" can be applied after selecting representative sub-basins for measurement.

If the total length, Σc, of the contours of the vertical interval, h, in feet, crossing the basin is measured by an opisometer, then $\frac{A}{\Sigma c}$ is the average distance between contours measured horizontally, and the average slope of the land surface is:

$$S_g = \frac{h \Sigma c}{A}.$$

To determine by opisometer the length of contours on a large area, even at 100-ft. intervals, is very laborious. Although the "sample area method" may be applied, it is better to use what may be called the "intersection method", which gives a fair determination of the slope for all parts of the area with moderate labor. To apply the "intersection method" the portions of topographic maps covering the area are divided by pencil base lines into, say, 4-in. squares. Choosing a suitable contour interval, the number of times, N, that the base lines are crossed by contours is counted and the total length, L, of base lines within the area is determined. If the contours crossed the base lines at right angles, the average slope would be $\frac{h\,N}{L}$. The contours, however, cross at varying angles from zero to 90 degrees. If x is the distance between the points where two adjacent contours cross a base line, and a is the angle between a normal to the contours and the base line, then the distance between contours, measured normally or at right angles to the contours, is $x \cos a$. Hence, the average normal distance between the contours is:

$$m = \frac{N \cos a}{L}.$$

The average slope of the ground surface is:

$$S_g = \frac{h}{m} = \frac{h\,L}{N \cos a} = \frac{h\,L}{N} \sec a$$

The average secant of angles from zero to 90° is 1.57; hence:

$$S_g = 1.57 \frac{h\,L}{N}$$

The slope ratio, $\frac{S_c}{S_g}$, may now be determined. This is never greater than unity. In other words, the slope of the ground surface of a drainage basin is always greater than that of the streams. The average length of overland flow or the distance which the water must travel on the ground surface before reaching a definite stream channel is an important factor in relation to flood discharge, especially for flat, poorly drained areas. The larger this distance the greater is the time of rainfall concentration at any given point on the stream. If α_g and α_c are the slope angles for the ground surface and stream channels, respectively, then:

$$\tan \alpha_c = \frac{S_c}{5\,280}$$

$$\tan \alpha_g = \frac{S_g}{5\,280}$$

Given these values, the average direction of overland flow or the average angle, z, between the direction of overland flow and the channel direction, can be determined by the formula, readily derived by geometrical construction:

$$\cos z = \frac{\tan \alpha_c}{\tan \alpha_g} = \frac{S_c}{S_g}$$

In other words, the slope ratio is the same as the cosine of the angle between the direction of overland flow and the stream direction.

The reciprocal of the drainage density, or $\frac{l}{d}$, is the average distance between streams. As there is a divide between every two adjacent streams the average distance from the divide to a stream is one-half this, or:

$$\frac{w}{2} = \frac{l}{2\,d}$$

The average distance which water must travel overland from a point on any divide between streams or the average length of overland flow can be derived from the relation:

$$\frac{w}{2\,l_g} = \sin z$$

from which,

$$l_g = \frac{w}{2 \sin z}$$

The writer has applied these and certain additional methods to various drainage basins for the purpose of providing data for the study of the relation of run-off to slope and other physiographic factors. It has been deemed worth while to present the methods in some detail here in order to lay a foundation for attacking the problem of the laws of flood discharge for different areas on a quantitative basis.

When a sufficient body of flood discharge data has been accumulated, accompanied by physiographic and soil or infiltration data, a beginning can be made toward the derivation either of a "rational" flood discharge formula, or perhaps better, the use of the existing rational method can be extended to large natural areas. To accumulate such a mass of data may require a long time, but the Engineering Profession worked for fifty years with the Kutter and similar formulas before reaching the point where the data for their application could be considered reasonably complete.

Flood discharge data unaccompanied by adequate data for the correlation of physiographic and other factors with the discharge coefficients are little more useful than a heterogeneous table of values of Kutter's n unaccompanied by data on the kind and character of surface for which the given values of n were determined.

Viewing the subject of maximum flood discharges from another angle, the writer believes that there is for each drainage basin a certain finite rate of flood discharge which Nature is incapable of transcending, that is, the true maximum discharge. Reasons for this belief are founded on: (1) Experience; and (2) meteorological considerations. The Hudson River cannot produce a Mississippi River flood for very much the same reasons that an ordinary barnyard fowl cannot lay an egg a yard in diameter—the hen is not large enough.

Rain can only be produced as fast as moist ascending air currents can be brought over the area on which the rain falls. The larger the area covered by a storm the smaller in general are the barometric gradients around the storm center. With less wind velocity and less rate of ascent of moist air the average rainfall rates over large areas must necessarily be less than may prevail over small areas.

On the other hand, in purely convective storms where the air ascends vertically, the moisture supply must in general be local. There is thus a natural inhibition of rain intensities exceeding certain limits, the limit depending on the size of the area. In view of these considerations a flood discharge formula such, for example, as the Fuller formula which implies that a flood of infinite magnitude may occur on any area, however small, if one only waits long enough, is, to say the least, irrational in form.

The question naturally arises: Is it possible to determine, even with approximate certainty, the true maximum or limiting flood discharge for any area? The writer believes it is possible provided a good record of flood discharges is available.

Flood discharge data when plotted in terms of frequency take the form of a skew frequency curve to which the ordinary or Gaussian law of error does not apply. Skew frequency curves, however, can be represented accurately

by other forms of expression. For floods the frequency equation should be of such form that the flood magnitude approaches a finite limit as the "exceedance" interval approaches infinity.

The following expression was developed by the writer to meet this requirement and has been applied to a variety of phenomena dependent on rainfall, including floods:

$$R = R_g \left[1 - e^{-kt^n}\right]$$

in which,

R = ratio of the magnitude of the event to its average magnitude.
t = average "exceedance" interval of an event of magnitude, R.
e = base of Naperian logarithms.
R_g = true maximum or limiting value of the magnitude of the flood or other phenomenon.
k, n = constants.

Values of k, n, and R_g can all be determined from the lower definitely located portion of the "exceedance" interval curve.

Data of flood records for 70 years for the Connecticut River at Hartford, Conn. (1843-1917), give the equation:

$$R = 1.82 \left[1 - e^{-0.255\, t^{0.54}}\right]$$

The highest recorded value of R is 1.70, but there have been nine floods with R exceeding 1.40. In other words, floods having magnitudes of more than 75% of the limiting flood magnitude occur at fairly short intervals, in this case about once in eight years.*

* Application of this form of frequency equation to the determination of maximum and minimum annual rainfall in New England will be found in the *Journal*, New England Water Works Assoc., Vol. 38, March, 1924.

FIRE-BANKS FOR OIL STORAGE

Discussion*

By Messrs. H. L. Shoemaker and Frank A. Epps.

H. L. Shoemaker,† Esq.—The practical results and conclusions at which the author has arrived, have not only been approved by the entire petroleum industry of the United States, but they have been used in a very practical way.

Mr. Hall's report was first presented at the Convention of the American Petroleum Institute in December, 1924, at Fort Worth, Tex., where his work was commented on very favorably by the oil industry. A committee of which the speaker was Chairman, recently collected and compiled data in regard to the hazards of petroleum products in storage, and found some astounding figures. The Committee was also able to use a great deal of Mr. Hall's data, especially in relation to the problem of the spacing of above ground petroleum storage tanks.

Regarding the fire record of petroleum products in storage in the United States, extending over a period of ten years (1915-25), eighty-seven oil companies, including all the large companies operating in the country, submitted the following information covering their fire history: In this period, 143 refineries experienced 73 fires, or 1 fire per plant in 20 years. In 11 132 marketing stations (ordinary bulk distributing stations where petroleum products are received by tank car, unloaded into storage, and distributed by tank truck), which embrace the territory from the Pacific Coast to the Atlantic seaboard, inclusive, and many of which are within the corporate limits of cities and built-up communities, 19 fires were experienced in the 10-year period mentioned, or 1 fire per plant in 5 860 years.

Regarding large seaboard terminals and pipe-line terminals, 273 such properties reported 77 fires in the 10-year period, or 1 fire per plant in 35 years; and 713 tank farms, probably representing a storage capacity of 500 000 000 bbl. of crude oil, experienced 88 fires in the same period, or 1 fire per plant in 81 years.

The oil industry feels that this constitutes a very remarkable fire record. Expressing the statistics in other terms and establishing a fire frequency ratio, or the number of fires in tankage compared with the number of tanks in service, it is found that 15 873 all-steel gasoline tanks had 41 fires in the 10-year period, or a frequency ratio of 0.026% per year, and that 12 594 all-steel

* This discussion (of the paper by H. H. Hall, M. Am. Soc. C. E., published in May, 1925, *Proceedings*, and presented at the meeting of June 3, 1925) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Gen. Eng. Dept., Standard Oil Co., Elizabeth, N. J.

kerosene and refined oil tanks experienced 35 fires in the same period, or a fire frequency ratio of 0.028% per year.

It is interesting to compare these annual fire ratios, or percentage of tanks lost by fire per year, with similar figures covering dwelling houses. The number of dwelling houses in the United States in 1920* was approximately 20 697 204. The number of fires in such dwelling houses per year† was 131 035, or expressed as an annual fire loss ratio, or percentage of dwellings burned per year, equals 0.63%, as compared with petroleum tank ratios mentioned of 0.026% and 0.028% for gasoline and refined oil products, respectively.

In view of the fact that the petroleum industry was vitally concerned with possible State and municipal regulations regarding the handling and storing of petroleum and its commodities, which might govern the construction of tanks, fire-walls and other requirements, the statistics given were of immense value. The Committee reporting on this work to the industry mentioned previously, in making recommendations regarding spacing of tanks, found Mr. Hall's work of immense practical value. To illustrate, in spacing crude oil tanks it was determined that a tank should be at least two diameters from shell of tank to adjacent property line in order to protect such property in case of fire. One of the essential reasons for recommending this distance was that Mr. Hall's data indicated that to be fully effective, a fire-wall must be placed approximately one diameter from the shell of the tank. Further, most ordinances specify capacity as a requirement in fire-wall construction around oil tanks. As shown by Mr. Hall's paper, capacity of a fire-wall is only one factor. The horizontal distance of the fire-wall from the container, as well as the type of construction used (that is, the kind of coping or lack of coping), are of more practical importance in protecting life and property and in retaining "boil-over" waves than mere capacity inside the wall or dike.

FRANK A. EPPS,‡ ESQ.—The oil industry has been intensely interested in the valuable data presented in this paper and also in a supplementary investigation conducted by Mr. Hall.§ While this paper deals with fire-bank protection required for oils possessing "boil-over" characteristics, the fire-bank requirements for tanks containing oils not possessing "boil-over" characteristics are also of interest.

Mr. Hall has stated, and it is a well-known fact, that refined oils do not "boil over". Clearly then, for this type of oil storage, the fire-bank can serve only as a secondary container in case of spills, leakage, or rupture of the tank. Except under unusual conditions, the oil industry has very strongly questioned the necessity or advisability of fire-banks solely for such purpose. Fire-banks capable of retaining at least the entire tank contents cost a great deal in proportion to the remainder of the installation in which refined oils are usually stored. It is questionable whether the benefit of protection obtained from this

* "World Almanac", 1925.

† "Safeguarding America Against Fire", December, 1924.

‡ Mgr., Fire Protection Dept., Tide Water Oil Co., New York, N. Y.

§ *Mechanical Engineering*, July, 1925, p 540.

expenditure is worth the cost. To answer this and other questions, a committee of the American Petroleum Institute was appointed to secure and analyze the complete tank-fire record of the oil industry in the United States for the 10-year period, 1915-25. The following is quoted from the report:

"* * * fire-walls surrounding gasoline and refined oil tanks were called upon to prevent the spread of fire in but 8 cases in 10 years. In 12 other cases fire spread because of the non-existence of fire-walls. Thus, fire-walls were effective or could have been effective in 10 years in only 20 instances. It is conservatively estimated that the fires reported represent the fire experience of not less than 50 000 steel-roofed tanks in service. Thus, fire-walls surrounding these 50 000 tanks could have been of practical benefit in only 20 instances—this in a 10-year period—or 2 per year.

"The Committee believes that this record of experience demonstrates the injustice of requiring elaborate fire-wall construction and the needless expense caused thereby, for the protection of this class of storage."

With respect to fuel oils, this Committee was able to obtain a record of only fourteen fires, in only one of which would a fire-wall have been of any practical benefit. The Committee felt, therefore, that the danger of fire in fuel-oil tankage is so extremely low as not to justify the cost of any fire-bank protection whatsoever.

Regarding fire-banks for crude oils, it is a peculiar fact that aside from the petroleum industry itself, fire-protection circles have regarded fire-banks merely as secondary containers, the function of which is to hold the contents of the tanks surrounded in case of spills, leakage, or rupture. Considerations affecting the fire-bank's capabilities of withholding a wave of burning oil thrown out of a tank by a "boil-over" have not received a great deal of attention; this, in spite of the fact that it has been generally recognized that the average fire-bank was not effective under this condition. Fire-protective legislation and insurance requirements to date have considered only the capacity of the enclosure and the material of which it is built, capacity, the chief specification, being usually 150% of the tank contents. The research work conducted by the Standard Oil Company of California under the direction of the author has clearly demonstrated that capacity alone is not a measure of fire-wall effectiveness when the tank contains crude oil.

The Committee of the American Petroleum Institute reached the conclusion that the bank around a crude oil tank must function both to throw back the wave of burning oil and to contain the liquid ejected. The usual "boil-over" from a crude tank seldom exceeds 10% of the tank contents. On the other hand, there may be several "boil-overs" during the same fire, also records are available wherein "boil-overs" are reported to have ejected more than 10% of the tank contents. The fire-bank must contain or hold this oil, not to mention large quantities of water often thrown upon the sides of the burning tank to keep it cool. The Committee felt, therefore, that a definite capacity specification should be provided. After investigating the research work performed by the Standard Oil Company of California and likewise

considering the tank-fire record of the United States for the last ten years, the Committee reached the conclusion that:

"Above ground, gas-tight containers for crude oils should be surrounded by fire-walls or dikes of earth or other suitable material, and should be:

"(a) Of a capacity equal to that of the tank or tanks surrounded, unless adequate drainage to a sump of like capacity is provided;

"(b) Located not nearer to the shell of any of the enclosed tanks than the diameter of the largest tank enclosed, and in any case not less than 50 ft.;

"(c) Provided with a suitable coping projecting inward to turn back boil-overs."

Mr. Hall's research work had a great deal to do with the formation of that recommendation, of which a General Conference of Oil Industry Representatives has voiced approval. The industry has accordingly recommended to the National Fire Protection Association that its model ordinance for the regulation of oil storage within corporate limits be revised to include such provisions for fire-bank design. The oil industry is obviously very appreciative of this fine piece of research work by the Standard Oil Company of California, and by Mr. Hall in particular.

REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

Discussion*

By Messrs. William G. Atwood, F. P. Shearwood, Jacob Feld, C. A. P. Turner, Walter H. Wheeler, Chandler Davis, R. W. Gausmann, E. E. Bauer, and Hardy Cross.

William G. Atwood,† M. Am. Soc. C. E. (by letter).‡—The Joint Committee is to be congratulated on the extremely valuable piece of work which has been done. There are, however, one or two parts of the specifications which it seems to the writer should be amended.

The specifications are written for use with Portland cement and certainly represent the best practice thus far developed for that material, but they should show on their face that they are written for Portland cement. This is particularly important on account of the increase in use of high alumina cement, now being manufactured in the United States, and the cements, known as "blended cements" for want of a better name, that are used in sea water. The writer, therefore, believes that on the title-page (page 1161§) the words "Portland cement" should be inserted to make the title read "Standard Specifications for Portland Cement Concrete and Reinforced Concrete". The same applies to the heading at the top of page 1166.§

In the definitions for "Aggregate", "Concrete", and "Mortar", the words, "Portland cement", are used. If the title is not changed as suggested, these definitions are incorrect, and the word, "Portland", should be deleted from each of them. This change will make them correspond with the definitions adopted as standard by the American Railway Engineering Association.

Section 88, entitled "Protection", does not seem adequate. In highly concentrated alkali waters the sulfates are much stronger than in sea water and there seems no reason why the protection provided in the specifications should not be as great or greater than for sea water as specified in Section 84; in fact, it would seem better if the first two sentences in Section 84 could be substituted for all of Section 88.

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* This discussion (of the Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, published in October, 1924, *Proceedings*) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., New York, N. Y.

‡ Received by the Secretary, October 10, 1924.

§ *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions.

The Appendices, of course, are reprints of the Specifications of the American Society for Testing Materials, but the same ambiguity exists in some of the titles as that already indicated. The title of Appendix XVI, for example, should read, "Proportions for Portland Cement Concrete of a Given Compressive Strength at Twenty-eight Days". As this title now reads it is incorrect if a high alumina cement is considered, and the same statement applies to the title of Appendix XVII.

These suggestions should be considered by the Joint Committee and the changes indicated should be made because they will correct the specifications and indicate the material under consideration without the ambiguity which exists at present.

F. P. Shearwood,* M. Am. Soc. C. E. (by letter).†—A study of these specifications seems to show that there is a lack of agreement between them and other specifications governing structural design. This must lead to much inconsistency and will not give relatively safe results for the different types of material.

In this specification, there are many sections indicating safe allowances and stipulating methods of calculation that are far from harmonizing with the specifications for other materials, particularly those for structural steel. For example, in accordance with Sections 168 and 169, a column enclosed in, or enclosing, concrete is allowed to carry a far greater load than if it is classed as a structural steel column and designed in accordance with the specifications for structural steel.

Again, in Section 167, the fiber stress in a column from bending, when combined with direct stress, may be increased by 20 per cent. The reason for this increase is not apparent, as the bending would generally be from the unbalanced live load. Furthermore, it will occur most frequently in cases where the stresses may alternate. In structural steel specifications, the fiber stress allowed for combined bending and direct stress from primary loading is always taken at the same unit as the direct compression, and these specifications also provide for the possible deteriorating effect of stress reversal.

The concrete specifications allow the full continuity of beams subject to bending, without restriction for possible settlement or deflection in the columns supporting them. This differs greatly from the conservative practice prompted by the structural steel specifications, and points to the concrete specifications having been prepared from a different point of view from specifications for other materials.

The safety of specifying unit stresses for building construction without any reference to the loading, seems doubtful. For instance, is a unit stress of 0.4 f_c (Section 188) correct for a hotel floor with a specified live load of 100 lb. per sq. ft. as called for by some cities, or is it a safe unit for a live load of 40 lb. per sq. ft. as specified by many other cities?

The inconsistencies between specifications for different materials must result in different factors of safety. This points to the advisability of revising

* Chf. Engr., Dominion Bridge Co., Ltd., Montreal, Que., Canada.

† Received by the Secretary, October 27, 1924.

the present grouping of the regulations which govern the designing and building of engineering structures. The material, workmanship, detailing, etc., which are peculiar to each construction, should form one group with separate specifications for each type of construction. On the other hand, each class of structure, according to its use, should have a specification to control the loads, formulas for designing, unit stresses, and other matters which are common to all materials, or which should give relatively equal results.

JACOB FELD,* ASSOC. M. AM. SOC. C. E. (by letter).†—These specifications constitute the most advanced and complete code of regulations for the use of concrete as a material of construction ever formulated, yet there are several items which are open to criticism both from a theoretical and a practical point of view, the modification of which, in the writer's opinion, will lead to a more general acceptance of the specifications.

According to the definition, it is a necessary condition that concrete contain Portland cement. This narrow definition of "Concrete" necessitates the formulation of some new names for the corresponding mixtures of aggregate, water, and non-Portland cements. Of course, the use of natural cements is relatively decreasing (although the absolute volume of natural cement produced and used is remaining fairly constant), and this cement is used for such unimportant structures that detailed specifications are hardly necessary. The rapidly increasing use of aluminum or electro-cements, however, must be taken into account. From the strides which these rapid-hardening cements have made in the construction industries in Europe, it may be expected that Portland cement will not be the only commonly used cement in the United States within a few years. Will these cements be used for concrete, or for some new-named building material? The writer favors a broader definition of "Concrete".

In defining the ratio of reinforcement as "the effective area of the reinforcement * * * to the effective area of the concrete" and the effective area of concrete as the "area of a section of the concrete which lies between the tension reinforcement and the compression surface of a beam or slab", it should be noted that the definitions do not apply to T-beams. The effective area of a T-beam in shear is a part of the area of the stem, whereas for flexure, the effective area is the product of the assumed width of the flange by the depth of the reinforcement for tension below the compression surface.

Beams along a discontinuous edge of a panel are variously named as lintel or spandrel beams when part of, or supporting, the exterior walls and also as marginal beams when either exterior or interior. These specifications classify all exterior beams as "wall beams". In Chapter XI on "Flat Slabs" (Sections 142 to 159), mention is also made to marginal beams, which may be either exterior or interior. Although not very important, a more rigid classification is desirable.

10.—*Mortar Strength Test.*—Standard Ottawa sand does not pass the specification for fine aggregate, and no one would consider using it as such

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† Received by the Secretary, November 1, 1924.

(disregarding the cost). It, therefore, seems that the mortar strength test for a fine aggregate should require more than 100% of the corresponding strengths of mortar made with standard sand, instead of the recommendation (in the foot-note) "it should preferably be 100". The minimum requirement in a specification has a tendency to become also the maximum.

28.—*Measurement of Aggregates.*—The "inundation" method of accurately measuring sand volumes has received so much favorable comment in the technical press that it deserves mention, at least as an alternate method, in the specifications.

Sections 29 and 30.—It is now well known and agreed that the minimum quantity of water should be used. Therefore, the repetition of the same statement in succeeding paragraphs is hardly required. If the last paragraph in Section 29 is omitted (as may often happen, when no tests are required), the statement "The quantity of water used shall be the minimum" ends Section 29 and begins Section 30.

Sections 42, 59, and 97.—Some definition of when "concrete has thoroughly hardened" seems to be necessary.

Section 42 requires that the concrete be kept at 50° Fahr. for at least 72 hours after placing or until the concrete has thoroughly hardened.

Section 59 requires that the forms shall not be disturbed until the concrete has adequately hardened, and the foot-note thereto states that "the proper time for the removal of forms shall be determined by the Engineer".

Section 97 states that the forms must be removed and the scrubbing for a rubbed finish performed before the concrete has hardened. The foot-note to this Section advises that in warm weather this will require from 6 to 24 hours and in cold weather from 1 to 3 days.

By combining these three references, the Engineer learns that "concrete has adequately hardened" from 6 to 72 hours after the pouring, the exact time depending on the temperature. The time of hardening also depends on other factors, an important one of which is the consistency of the mix, and some definite method should be specified which will determine when concrete has adequately hardened without leaving it to the judgment of the Engineer (who cannot be expected to be at hand to make the decision at any time), or to the Inspector who may represent the Engineer on the job.

Section 51.—One of the methods for depositing concrete under water is defined as the "Drop-Bottom Bucket", which cannot be dumped until it rests upon the surface upon which the concrete is to be deposited and the bottom doors of which, when tripped, will open freely downward and outward. This is hardly possible and the description requires a slight modification.

Section 53.—It seems to the writer that the specifications dealing with "Forms" are lacking in the entire omission of the subject of steel forms. The almost general use of metal forms for circular columns and the recently developed panel forms for beam and slab as well as flat slab construction, warrant an inclusion of other than wooden forms in the specifications. It is interesting to note that the newest type of metal forms are made so as to give a perfectly smooth surface finish; in some cases, the form manufacturers claim that

the ceilings require no further treatment, eliminate the necessity for plaster where now used, and are smooth enough to take paint directly. If these claims are substantiated, the use of metal forms will considerably increase.

55.—Workmanship.—The writer is glad to note that wire ties are not permitted except in light and unimportant work. Some time ago, he had occasion to inspect some concrete basement walls, 16 and 18 in. thick, built in a coarse sand and gravel soil to a depth of 8 ft. below grade, but probably several feet above natural ground-water level. Wire ties had been used to keep the wall forms in place. Four months after the concrete had been poured, rust streaks appeared on the inner faces. The walls had received no surface treatment on either face, and the rust streaks appeared after every rain for about six months thereafter, especially at the wire ties in the upper part of the wall, no other indication of moisture seepage appearing elsewhere. Either the ties have rusted out completely or the pores have become naturally clogged, for the discoloration has stopped. The writer would prefer to see the use of wire ties prohibited entirely.

64.—Splicing.—It is common practice in splicing bars to tie the lapped lengths together. The Inspector will find considerable opposition if he tries to enforce the specification that the bars be separated by the minimum distances specified. The specifications are, however, entirely correct.

66.—Future Bonding.—Rods left exposed for future bonding should be protected from corrosion, but should not be coated with an oil paint. A rusted rod is much better than a painted one, because paint is difficult to remove thoroughly. It might be advisable to specify a coat of mortar on all exposed rods. It is surprising to note how often specifications call for paint on structural steel which is to be encased in concrete. A section should be included in these specifications prohibiting this practice.

69.—Joints.—Where additional resistance to horizontal shear is required the specifications recommend the use of partly embedded stones at joints. A more usual, and probably cheaper, method which the specifications disregard, is the provision of dowels or steel rod bonds which are much better in thin walls than projecting stones and certainly just as efficient in providing shear resistance.

74.—Expansion Joints.—The necessity of providing for expansion and contraction in long buildings is often disregarded, because of the cost and difficulty of constructing double columns and expansion joints in slabs. In long buildings several stories high, the cost can be decreased considerably by providing expansion joints in the top one or two stories only, as the temperature range is greatest at the roof and comparatively small at the lower stories.

78.—Integral Compounds.—The Committee is to be commended on its clear-cut decision against integral water-proofing compounds.

83.—Depositing.—It seems very difficult for "practical engineers" to comprehend the true meaning of an integral concrete mass and that a continuously poured surface is much better than one containing several pour lines, whether or not the several pourings are bonded together. The writer recently saw a design for a concrete pier, to be built in a stream exposed to the action

of warm tidal currents as well as a cool mountain stream, the pier to be built by a method similar to that of concrete frame building construction. A concrete frame of columns and struts is first built on the shore and, later, the open panels are poured when the frame has been floated into position, the formwork enclosing the frame providing the necessary buoyancy. In spite of the fact that the forms are to be left in place to protect the concrete during the setting and hardening period, the concrete eventually exposed to the sea water will not present a homogeneous surface to the action of the water and the crustacean agents tending to disintegrate the concrete. The advantages claimed for the method adopted over the design of a monolithic continuously poured concrete pier was estimated lower first cost and ease of construction, especially the latter.

The design specifications given in Chapter XI cover the field very thoroughly, several important items usually not considered are carefully covered. Section 112, "Unsupported Flange Length", is stricter than the usual design practice; rectangular beams carrying partitions between shaft openings are often made the width of the partition without regard to the span. Section 112 restricts the use of a 6-in. beam (wide) to a 12-ft. span and an 8-in. beam to a 16-ft. span. These sizes are often used for spans of 20 ft. or more, although the designer will admit that such practice is theoretically not sound.

115.—Flange Width.—The flange width of T-beams is restricted to one-half the clear distance between beams. This is true for beams of equal sizes only. Where a heavier beam parallels a lighter one, a greater part of the slab between them acts with the heavier one, and this should be taken into consideration.

119.—Isolated Beams.—The writer can see no justification for the requirement that an isolated T-beam shall have a flange thickness not less than one-half the width of the web. This case seldom arises, the beam being usually quite wide. This requirement practically forces the beam to be designed with the neutral axis in the flange. If this is desired, it should be definitely stated. As the section reads, the economical design is a narrow beam.

142 to 159.—Flat Slab Design.—The design of flat slabs is far from a perfect theoretical solution. It is hardly necessary, therefore, to require the use of such complicated formulas for its design. The specifications, if followed completely, complicate the method of design to an unnecessary extent, providing excellent chances for error by the unexperienced designer. Every slab is considered as rectangular. Although most flat slabs are for almost square panels, very seldom does an exactly equal column spacing occur in both rectilinear directions. It seems advisable, therefore, to permit the use of an average span and the design of the panel as if it were square, within certain limits of difference in the lengths of the two sides. These specifications have made no such provision, nor have they restricted the use of the formulas to slabs in which the greater width is some ratio, say, 4 : 3, to the smaller.

The value, W, is open to various interpretations. It is probably meant to include the weight of the drop panel (when such is used), although the speci-

fications do not so state definitely. In a rectangular panel, should the *W* be taken as the actual weight of the panel, or of an equivalent square, to be used for the moments in each direction (as required by the New York Building Code), or should a *W* corresponding to a square panel of the span considered, separately for each width of the rectangular panel (as required by the Chicago Code), be used? The interpretation decides the economy of the design, assuming that both codes are formulated to require the same factor of safety (which, of course, is not true). The interpretation in the New York Code requires less total reinforcement.

The design of an exterior panel according to these specifications makes the reinforcement at right angles to the wall, and, therefore, also the stiffness in that direction, so much greater than in the direction parallel to the wall, that practically all the panel load will span in the former direction. The design does not seem to be balanced, nor does the steel required seem necessary to one who is accustomed to the design of flat slabs according to the New York Code. Along the column line parallel to, and one panel from, the wall, the negative moment on the column strip is increased 15% and that on the middle strip 30%, as against a 20% increase in each in the New York Code. The same corresponding increases are called for in the positive moment at the center of the exterior bay. Nothing is definitely provided for the negative moment at the wall, but the rods used for positive reinforcement must continue to the wall. This is equivalent to the assumption of a simply supported slab at the wall. Compared with the present method of the New York Code of providing for no positive moment near the wall, although some steel is often carried in the bottom of the slab to the wall, it will be seen that the present specifications (New York Code) require less steel, and, at the same time, approximate the actual condition of a slab poured monolithic with the columns and marginal beams than the specifications of the Joint Committee. As the specifications read at present they are applicable to the case of a slab resting on a brick wall, but not to the usual case of a flat slab in a concrete building, and, yet, Section 149 intimates that some negative reinforcement should be provided normal to the wall, in addition to the full positive reinforcement.

148.—Panels with Marginal Beams.—A marginal beam, because of its greater stiffness than the adjoining slab, must carry a greater load per unit width in order to receive the same deflection. The portion of the load taken by the beam depends upon the relative rigidity of beam and slab. A beam with a depth no greater than the dropped panel is assumed to take no extra load; and as soon as the depth of the beam becomes greater than that of the panel, even if it be a bearing wall (Section 150), it must be designed to carry 25% of the entire panel load. This is equivalent to 50% of the load carried in a direction parallel to the beam, assuming a square panel. For the usual case, this seems excessive, and far from logical. A bearing wall may effect so great a change in the normal deflection of the slab as to cause the relief of such part of the load from the slab, but a shallow beam certainly cannot act in the same way. It seems that a simple relation can be

specified to determine the portion of the panel load to be carried by the beam depending on its stiffness. Otherwise, if the specifications are rigidly adhered to, marginal beams will tend to become just as deep as the dropped panel (which may be advisable in spite of the resulting increase in deflection), except where the fire insurance requirements insist on an exterior beam with a soffit projecting at least 12 in. below the under side of the slab.

153.—Arrangement of Reinforcement.—The provision requiring a number of double bent rods in each band will receive opposition from several builders on the grounds that such rods are too long and heavy for easy handling and that single bend bars are just as efficient. There is no theoretical basis for the requirement of such long rods. Most designers realize that the line of inflection is only an assumption and that it fluctuates with variations in loading. These specifications require that every rod be extended 20 diameters beyond the line of inflection to provide for such variation. A cheaper method is to make all rods of the same length, to extend from inflection line to inflection line, and then stagger both the ends of the rods and the bends, say, 6 in. each side of the assumed line of inflection. Investigation will show that the small moment requirements near the line of inflection are then provided for both in bond and in tension. In general, the "job" prefers a design which gives the fewest different lengths of rods, and the fewest number of rods, provided the size is such that they can be easily placed.

160 to 171.—Columns.—The specifications seem to favor the spiral reinforced column as against the plain rodded column. By permitting the use of spiral reinforcement as low as ½% and vertical steel as high as 6%, the range of available use of the spiral column is increased as compared with the limitations imposed by the New York Code, whereas the rodded column is decreased in use by the restriction to 2% maximum vertical steel. The spiral column, if square, requires only 1½ in. of concrete protection; the rodded column requires 2 in. Lateral ties in columns cannot be spaced more than 8 in. apart. Rigid adherence to these specifications means the practical elimination of the column without spiral reinforcement, for no just reason because the plain rodded column has shown up as favorably in tests as the spiral type.

173.—Soil Footings.—In determining the load per unit area on the soil, there is no reason for omitting the weight of the footing itself, which dead load should be added to the column load to obtain the true load on the soil. This item is very large in wall and combined footings.

175.—Sloped or Stepped Footings.—It is quite easy to specify that "sloped or stepped footings shall be cast as a unit", but not so easy to build such a monolithic footing. This should be taken into consideration and proper bonds or dowels called for to tie the successive plinths together. One type of footing very often used where good rock is not too far below grade is the steel tube filled with concrete. Although really acting as a column and not as a pile, the ordinary column design formulas do not apply and some special rules should be included in the specifications to cover this type of construction.

Another omission from the specifications is the design of concrete stair slabs which are often made so thin that they seem to defy the laws of gravity and persist in remaining in place by "force of habit".

Unit Stresses.—There may be some objection to the higher unit stresses allowed for concrete, but the writer fully agrees that a working stress of 800 lb. per sq. in. is not too high for a concrete that actually tests 2 000 lb. per sq. in. He would also like to see some allowance made for the tensile strength of concrete, especially when little steel reinforcement is used. He would like to raise the question of what should be done in the case of a building designed and constructed on the assumption that the concrete would test 2 000 lb. in 28 days and the tests actually showed that the concrete was much weaker.

C. A. P. Turner,* M. Am. Soc. C. E. (by letter).†—The specification of the Joint Committee presents a slump test for consistency and leaves the temperature of the material entering into the mix indefinite. Sand and stone in the stock pile are often so heated in the long summer days of the North that the mixtures may stiffen in the barrow or car before it can be dumped in place on the work, and cooling the aggregate by wetting becomes essential for workable consistency. The slump test thus lacks definiteness outside the constant temperature of the laboratory unless the temperature of the material entering the mix is given consideration and definitely stated.

Observance of Section 45 would make the cost of cyclopean masonry prohibitive. In filling a reinforced concrete shell for bridge piers, two-man rubble as plums may be dropped 40 or 50 ft. into a plastic concrete matrix and thoroughly bed themselves without labor, but such masonry, although economically meeting practical requirements, is barred without reason under these specifications.

Reinforced concrete construction in the larger cities is governed by building code rules or laws. The maximum unit stresses specified in the various codes present no radical divergence; but the rules of design for applying these working stresses differ so widely that a consistent correlation to the exact principles of applied mechanics is obviously lacking. In other words, building code rules are to a predominant extent "rules-of-thumb", based on assumption rather than on engineering analysis, substantiated by test data—or more unfortunately—in some instances deliberately arranged in detail to favor special commercial interests rather than the interests of the public.

If this specification is to replace the rules of ordinary city codes, it must be founded on a sound economic theory of development of the greatest strength at the lowest cost.

The specification for "Columns" falls far short of this desideratum. Why should "drawn wire" be preferred to wire rods for spirals, and what benefit arises from the increase in cost involved? On what test data is founded the diction that one-fourth the volume of verticals shall be provided as spirals in the hooped column when test results show this ratio to be too small?

* Cons. Engr., Minneapolis, Minn.

† Received by the Secretary, November 17, 1924.

In the specifications of the first Joint Committee, the percentage of hooping was fixed irrespective of the quantity of vertical steel. As maintained by the writer twenty years ago, the present rule recognizes that a definite relation between the hooping and vertical steel is required for the development of the maximum efficiency, but it fails to credit the hooping with the proportionate increase in strength found by test. Misconception of the significance of deformation test data is responsible for the pessimistic attitude exhibited in this rule toward the value of the hooping. Experiments at the University of Illinois disclose that the lateral swelling or bulging of the column with the increase of the load toward failure is relatively small at mid-length of the column. Therefore, the part played by the hooping was considered correspondingly insignificant without investigating other parts of the column in which the lateral expansion due to the stress distribution would be far greater than at mid-length. In a study of the stress mechanism of the solid column,* the writer finds that the direction of the maximum shearing stresses make only a slight angle to the axis of the column at mid-length, whereas at the fifth and quarter-points of the column the direction of these stresses approach 45°, so that the maximum lateral expansion occurs at that locus. Likewise, the solid column of such relative dimensions as the ordinary concrete column fails at about the quarter-point rather than at mid-length.

The formulas of the Joint Committee thus place a premium on the least reliable construction, the tied column, and discriminate against the safest type, the properly hooped and vertically reinforced member, and in this respect the proposed rules are open to the same criticism as some of the least progressive present-day building codes.

The Joint Committee states that the report of the Committee appointed in 1904 is used as a basis for the present specifications. According to the *Cement Age,* the 1904 Joint Committee was selected to avoid suspicion of commercial interest in devising rules for the safe and economic construction of reinforced concrete buildings and for that reason structural steel engineers and chemists were considered by the cement manufacturers and chemists to be better fitted for the formulation of ethical rules of design. A set of structural steel rules of design misapplied to reinforced concrete members was the natural outcome.

Structural steel rules for the design of reinforced concrete differ from those appropriate for the design of the natural types of reinforced concrete construction because of the divergent characteristics and utilization of the materials involved. Steel presents substantially equal resistance to tension and compression. Conversely, concrete has only 10 or 12% of the strength in tension that it possesses in compression. When strengthened to resist flexure by embedded steel, the steel furnishes the direct tensile resistance, the concrete the resistance to compression, shear, and indirect tension. Whereas in the steel beam the positive and negative horizontal shears are equilibrated at the neutral plane, in the reinforced concrete beam this equilibration takes place at the surface of the steel and engineers are dealing with an

* "Elasticity and Strength of Materials", Section IV.

entirely different stress mechanism, one in which for safety and economy shear distortion must be given careful consideration in contradistinction to the rectangular beam of homogeneous material, in which the shear strain may be disregarded as is done in the ordinary theory of flexure.

From the mode of manufacture, steel is suited best to unit construction in which the external moment may without serious error be assumed as a concrete quantity resisted solely in the vertical plane of the member by internal bending resistance about parallel horizontal axes.

Concrete, from its formation, may be most advantageously utilized as a monolith. Its flexural resistance may then be developed as demonstrated in the mathematical theory, in three planes at right angles, instead of in one plane only, as is practical with steel.

The flexural resistance of a well-designed monolithic slab floor thus becomes a composite of twisting and bending resistance and the applied moment of the external forces must be treated for rational analysis as an abstract quantity balanced or equilibrated by the algebraic sum of twisting and bending resistances according to the elementary laws of equilibrium demonstrated a century ago. These laws, however, are ignored in the Joint Committee's rules for the design of monolithic reinforced concrete floors.

Statically determinate or unit construction is used to a limited extent in reinforced concrete work. For such construction the formulas of the Joint Committee are open to criticism for shallow beams.

The resistance of a shallow beam of a span twenty-five to thirty times its depth, with 1% of metal, appears to be satisfactory when examined by the Committee's formulas notwithstanding that excess shear distortion may occur such as to check the member in an unsightly manner under working load, accompanied by permanent deflection and obvious weakness in developing the allowed working stress in the steel. This should be avoided by investigating the magnitude of the shear distortion and reducing it to a safe value, by limiting the allowable percentage of metal to a smaller quantity for the shallow beam than would be permissible for a deep beam.

The external forces of the load in the simply supported beam produce clockwise rotation looking at the left side accompanied by longitudinal compression of the upper fibers and stretching of the lower ones. The reaction of these positive and negative extensions is equilibrated by horizontal shear strains causing anti-clockwise rotation and tending to relieve the moment stresses accompanied by an increased deflection until equilibrium is restored at mid-span.

There are accordingly two distinct kinds of angular distortion of a square prismatic element normal to the face of the beam, that is, trapezoidal deformation accompanying the positive and negative elongations represented by shortening of both the diagonals of the square prism in the compressive zone and lengthening both diagonals in the tension zone, this kind of deformation being greatest at mid-span and zero at the end. The reaction to the longitudinal elongations in their equilibration by horizontal shear stress causes rhombic deformation, the stretching of one diagonal and the shortening of the other

diagonal of the elementary prism. This deformation is zero at mid-span and increases to a maximum toward the end. Obviously, the sum of these two angular deformations is productive of the maximum distortion and of the greatest diagonal tension which determines the locus and direction of cracking of the concrete by shear strain. As the trapezoidal distortion is small in the deep short beam, shear failure is produced predominantly by the rhombic deformation and its locus will be toward the end of the beam where that distortion is a maximum. Conversely, in the shallow beam, trapezoidal distortion will exceed the rhombic deformation and the point of maximum shear distortion and checking of the concrete will move from the end toward the center.

Building codes attempt to provide for angular shear strain in concrete by dividing the vertical reaction by the area of the beam, and obtaining what, by assumption, is called the unit shear stress in the concrete. From the nature of shear distortion and its variable position of maximum with different ratios of length divided by depth, this computation fails to indicate an approximate or relative magnitude of the angular shear stress which causes the concrete to crack. In fact, the error is so large that the failure ordinarily occurs in the shallow beam where the shear stress is not more than one-quarter the maximum as commonly but erroneously computed. The combination of the diverse angular distortions has been treated by the writer in a graphical manner,* so that the determination of the maximum may be made as easily as the irrelevant computations under the ordinary code.

Not only is the magnitude of the internal shear strain not proportional to the external shear force as ordinarily calculated, but it is, moreover, much smaller in the fully restrained beam than in the simply supported concrete beam of the same dimensions.

The rhombic component of the rotational shear strain is only one-fourth as great for a concentrated central load in the restrained beam as for the simple beam and only one-fifth as great for uniform load.

The greatest value of the trapezoidal component of the rotary shear distortion in the restrained beam is only one-half as great for a central load and two-thirds as great as it is in the simply supported beam for uniform loading. The combined maximum shear distortion is thus generally little more than one-third or one-half as great in the restrained beam as in the simply supported beam of the same section of concrete, yet ordinary building codes require the same section of concrete to resist the much smaller strain in the restrained beam that would be required for the much greater strain in the simply supported beam. Adoption of exact rather than approximate theory is the remedy for the uncertainty and waste involved. The work of design is complicated by meaningless computation and the reasonableness of procedure is so obscured that inherent weakness of design may often pass unnoticed.

Continuous statically indeterminate monolithic construction of beams and plates is commonly adopted for economy in reinforced concrete work, but the

* "Elasticity and Strength of Materials", Section II.

Joint Committee's formulas are not in accord with experiment for this type of structure.

W. K. Hatt, M. Am. Soc. C. E., quotes* tests of continuous beams and girders from the *Bulletin* of the Engineering Experiment Station of the University of Illinois, showing the ratios of percentage of load moment in steel stresses as given in Table 49.

TABLE 49.

	Steel stress, in pounds per square inch.	Percentage of load moment in steel stresses.
Beam	16 000	77.0
Girder	17 000	79.0
Beam	7 500	64.0

The percentage of load moment, however, disregards the interference of shear detrusions which reduces the maximum steel stresses without lessening the maximum compression in the concrete for a given curvature.

For a ratio of $\frac{l}{d} = 10$ in a continuous concrete beam, the tension in the steel at mid-span may be 30% less than that computed by disregarding this interference of detrusion, and for $\frac{l}{d} = 14$, it is approximately 15 per cent. This discrepancy is erroneously attributed to the tensile resistance of the concrete by those whose study of the subject has not extended beyond the approximate assumptions of the common theory of flexure.

The exact theory of flexure permits computation in harmony with experimental results, except as they may be affected by shrinkage stresses which disappear after one or two repetitions of load. A greater divergence for the column-supported slab appears between experimental measurement and the Joint Committee theory.

Thus, Professor Hatt tabulates† the percentage of the computed bending moment according to Joint Committee rules accounted for by measured steel stresses, as shown in Table 50.

From Table 50 the proportion of the moment accounted for by the steel is one-fourth or one-fifth what the Committee assumed it should be by its structural steel rules for the design of column-supported slabs.

In view of the close agreement between measured steel stress in the simple beam as compared with that calculated by the Joint Committee rules for slabs, a divergence of 300 to 400% warrants the presumption of error in the rules. This would be explained by the following statement* of Professor Hatt:

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1560.

† *Loc. cit.*, p. 1561.

"* * * it is evident that, even at steel stresses when the concrete is broken down in tension at some sections, there is a marked discrepancy between internal and external moment." (Amounting to 400 to 500 per cent.)

Quoting further:*

"The evidence at hand points, probably, to a more persistent operation of these tensile stresses in the case of flat slabs, with about one-half of 1% reinforcement, than in beam structures, which are more heavily reinforced. * * *

"Individuals will no doubt attribute these remarkable discrepancies between steel internal moments, appearing in steel stresses and external moments, to some other action than the tension in the concrete. The writer would rather look to the latter than to some poorly defined and unproved agency."

To be acceptable, this belief in the tensile resistance of concrete should be substantiated by experience and experiment. A slab, 21 ft. square by 5 in. thick, of excellent concrete, with 24-in. strips of chicken netting, four ways, supported by a column in the center and by four walls on the edges, carried very nicely a 7-ton test load of barrels placed in a circle between the posts and the wall, until the temperature dropped 23° at night, when the slab collapsed completely. Such a slab reinforced with $\frac{3}{8}$-in. rods on 4-in. centers each way, with the rods raised over the post, would carry 160 tons without over-strain or twenty-three times the load under which it collapsed, with not only the tensile resistance of concrete acting but the chicken netting as well. Professor Hatt's explanation, therefore, differs from experiment by twentyfold.

TABLE 50.

Building.	Load, in Pounds. per Square Foot.		Number panels loaded.	Positive.		Negative.		Total coefficient.	Per-centage.	
	Design.	Test.		Stress.	Coefficient.	Stress.	Coefficient.		Theory.	Joint Committee.
(1) Franks	250	739	2	10 100	0.0068	9 200	0.0149	0.0217	24.0	28.0
(2) Larkin	250	738	2	16 000	0.0090	8 500	0.0135	0.0225	25.0	29.5
(3) Shultz	300	844	4	6 200	0·0038	7 100	0.0105	0.0143	15.8	19.0
Average.....	....	......	....	....	0.0065	....	0.0130	0.0195	21.6	25.5

Unanimous belief in the efficiency of this assumed tensile resistance of concrete on the part of all the members of the Joint Committee has been claimed in testimony of record of two former members in patent litigation, and U. S. Circuit Courts of Appeal whose familiarity with elastic resistance was such that astonishment was expressed at the idea that a floor could bend under load, accepted this testimony, reversed the scientific decision of the Engineer Examiners of the U. S. Patent Office, and decreed for the advancement of

* *Transactions*, Am. Soc. C. E., Vol. LXXXII (1918), p. 1562.

this branch of scientific endeavor, the following legal facts in respect to concrete floor construction: That a flat arch is the same in principle as a continuous plate; putting the steel in the top of a reinforced concrete floor is the plain mechanical equivalent of putting it in the bottom; strips that do not cover the area are equivalent to crossed-rod reinforcement in plate action; and, finally, that radial and circumferential resistance is the same as radial in all directions. If the concrete worker should be guided by the legal facts, failures as serious as that of the Knickerbocker Theatre at Washington, D. C., would be a daily occurrence.

The derivation of the legal facts noted accords strictly with the American system of technical jurisprudence under which scientific facts are by judicial instinct to be smelled out of conflicting testimony and faultless logic afterward applied to the data thus secured in determining the finding.

The testimony that before the concrete cracks (the perturbations induced by the principles of rigidities notwithstanding) the mode of operation is identically the same whether the steel is in the top or the bottom of the slab in flexure, combined with the alleged experimental evidence that the tensile resistance remains operative after the concrete is cracked and broken by tension, justifies the logical derivation of opinion under the workings of American equity. These facts indicate that the missing link of evolutionary development to cover the gap between the sane and the insane in technical jurisprudence lies in the rational weighing of scientific fact by an intelligence skilled in the art, according to the decision in Arkwright *v.* Nightingale, C. P. 1775, that the specification is addressed to persons in the profession having skill in the subject matter and not to men of ignorance. (See, also, Carnegie *vs.* Cambria, 185 U. S. 403, 40 L Ed. 968.)

Although divergence in belief as to the relative tensile resistance of steel and concrete in the flat plate may obscure the elastic relations of moment to deflection and of measured to computed steel stress in the composite plate, this obscurity should disappear in experiments with the homogeneous plates and indicate the accuracy or inaccuracy of the design formula.

Take a celluloid plate, support it at opposite edges, and suspend the applied load on a string through a hole in the center of the plate, such as a 2-lb. load on a 12 by 12 by $\frac{1}{16}$-in. plate. The curvature is substantially cylindrical, but the deflection is four times as great as if the plate were supported on four sides, notwithstanding the applied moment is only twice as great. The halving of the internal moment stresses, which are proportional to the deflection, is caused by the torsional resistance erroneously disregarded in the Joint Committee's rules for the design of plates.

By omitting the torsional resistance of the plate, the attempted application* by John R. Nichols, M. Am. Soc. C. E., of the principles of statics cited by the Joint Committee results in computed steel stresses for the square plate freely supported on four sides twice those developed in the practical plate under a central load, and this divergence is increased in the continuous column-supported slab over that of the slab supported on four sides because in the

* *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1670.

latter the twist interferes with continuity whereas in the former it co-ordinates with continuity. The deficiency of the Joint Committee rules for the beam lies in the lack of an exact analysis of shear strain and the same deficiency is duplicated in its analysis of the plate.

St. Venant has demonstrated that, in the homogeneous prism acting as a beam, the stress mechanism changes little whether the load is uniform or concentrated, and because a central concentration presents a simpler problem for investigation than a uniform load, such a load and its travel to the support will be treated for the purpose of illustrating the kind of a stress mechanism with which the designer has to deal in the flat-plate floor reinforced by suitable mats of crossed rods.

Take the case of an indefinite series of column-supported square panels, each loaded at the diagonal center of the span with a concentrated load covering the same area as the column capital. For simplicity, treat the weight of the plate as very small in comparison with the weight of the concentrated central load so that for preliminary investigation it may be disregarded. By symmetry, one-fourth the central load must travel to each post; the concave area of the suspended span must by symmetry equal the area of the convex cantilever.

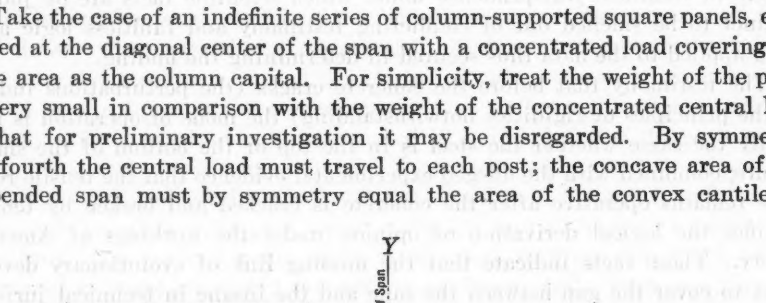

FIG 59.

Thus, in Fig. 59, one-eighth the load, C, travels to the column, A, through the triangular area, ABC. By symmetry, OB and OD are straight lines normal to the diagonal, AC. As DB by symmetry is unbent if the dead load is negligible, the weight of one-eighth C must be uniformly distributed in vertical shear along OB, which will be divided into four equal parts at Points 1, 2, and 3. As the phenomenon of buckling of thin plates shows that the load

travels in a straight line to its support, the planes of vertical zero shear dividing into equal parts the load, C, which travel to OB, will be represented by $1C$, $2C$, $3C$, and BC. In like manner, each of these four parts of the load, C, must travel between the vertical planes, AO, $A1$, $A2$, $A3$, and AB. To determine the planes of greatest twisting shears about the vertical axis we need as co-ordinates the curves of greatest vertical shear, the curves of zero vertical shear, and the locus of greatest twisting shear, about axes parallel to AC.

The twisting shear about AC, or parallel horizontal axes, is a maximum where the curvature passes through zero, just as the bending moment is a maximum where the vertical shear passes through zero, so that the straight line, OB, is the locus of the greatest twisting shear, and similar planes of greatest twisting shear will be parallel to OB and normal to OC at other points between O and C. By dividing OC into four equal parts and drawing $c' f'$, $b' c'$, $a' d'$, these planes are spaced at equal distances and circles may be drawn about C as a center, through a', b', c', and O. Because these circles are normal to the lines, OC, $1C$, $2C$, $3C$, which represent zero vertical shear, they will be the planes of greatest vertical shear and, accordingly, the co-ordinates are obtained by which the spirals of twisting shear may be drawn right and left through the intersection of the curves of zero vertical shear and greatest twisting shear about the diagonal, AC and through the intersections of the curves of zero vertical shear and greatest vertical shear. Then, the contours of curvature may be located through the intersection of these shear spirals and from O to W the curve bounding the area of the warped surface as distinguished from the dished areas of cantilever and suspended span.

A part of Fig. 59 shows the twisting shears on the top surface of the plate and in another part the twisting shears on the bottom of the plate, yet the intersection of these two different sets of twisting shears locate different points on the same contour of curvature which because the twist is equilibrated at the middle plane of the plate are principal stresses on that plane in plan.

The principal stresses on the top of the plate make a slight angle with the contours in one direction and on the bottom of the plate a slight angle with the contours in the other direction except where these stress curves cross the planes of principal curvature and the twisting shears change sign when they become coincident with each other.

In the saddle-shaped area at the boundary of the panel, the planes of principal curvature lie in the column and median lines, as along YY and AA'; and at 45° to these lines, the curvature of the saddle-shaped area changes sign and the inclination to the horizontal is greatest normal thereto. Along the diagonal, because the twisting shears there change sign, the stress mechanism in the vertical plane is similar to that of the continuous beam, and the plane of inflection, O, will be half-way between A and C, as it is in the restrained beam under concentrated central load if the dead weight of the beam is disregarded.

Consider the change which occurs by reason of the dead weight of the plate in addition to the concentrated load at C: The curvature of the saddle-shaped area along a column line and normal thereto would no longer be equal,

the plane of zero curvature and greatest twist of the saddle-shaped area would rotate from a 45° direction toward the column lines, and the point of inflection, *O*, would move toward the column. The planes of greatest twisting shear would no longer be normal to the diagonal, but would remain parallel to the plane of the greatest twist of the saddle-shaped area.

The contours thus determined agree with the contours found by experiment. The deflections computed from the combined twisting and bending and those indicated by this stress mechanism accord with those found by experiment. The twisting, increasing from the center of the dish-shaped area toward its margin, is in accord with experimental observation of the closing of the slits of the circular plate model. It shows that the equations of Lagrange are not in accord with experimental facts; that the assumption of a uniform Poisson ratio to represent the lateral squeeze is not in harmony with the angularity of the curves of twisting shear about the *ZZ*-axis, which define the contours, nor are the curves of greatest displacement in the compression zone coincident with the direction in which research workers measure the compression of the concrete in their field investigations. Having found the direction of the curves of twisting shear, the direction of the curves of principal stress may be determined from them and then the curves of displacement and equi-potential, by bi-secting the angles between these shear curves and the principal stresses. These curves differ from each other in the following particulars, as all writers on mathematical theory agree.

Principal stresses on an ideal plane of division, which traverse any given point of a body, cannot change suddenly either as to direction or magnitude while that plane is gradually turned in any way about a given point. A sudden variation can only take place at a surface where there is a change of material. On the other hand curves of displacement and equi-potential in vertical planes change abruptly, forming a cusp in passing from the compression zone to the tension zone, and in like manner they change abruptly on the surface of the plate where the direction of the twisting shears change sign. Thus, in anticlastic or saddle-shaped curvature with equal and opposite bending the twisting shears change sign at planes at 45°, or planes of principal curvature of the plate. If these planes have unequal curvature, however, the planes of zero curvature rotate toward the sharper principal curvature. The location of such a plane in the warped surface of the column-supported plate is the first step in the stress analysis in the continuous plate. The place where twisting shears change sign and neither curvature passes through zero determines the plane of greatest curvature of the plate. Such a plane in passing through consecutive points of the plate is the plane of greatest curvature for each of the points and therefore for the plate as a whole. Thus, in the column-supported plate of Fig. 59 the diagonals are the planes of principal curvature in the dish-shaped areas and the column and median lines the planes of principal curvature in the saddle-shaped area.

To the extent that the metal is distributed uniformly as a mat in the form of relatively small rods the composite plate or homogeneous plate may operate, as far as the mechanism of the twisting shears in the concrete is concerned, in

almost identically the same manner in the compression zone of the plate. If, however, the rods are not distributed in the form of crossed-rod mats, but have open areas and a concentration of metal on the diagonal and column lines, the rigidity of the strips induces anticlastic curvature about the diagonals as well as about the column lines, and the stress mechanism may be solved by following the same principles as has been followed in the uniform plate when it becomes apparent that the resistance of the metal degenerates into substantially that of beam action. For such a plate even the Joint Committee rules would be deficient in providing sufficient metal and the deflection would be many times greater than that of the practical and successful structure in present-day use.

That the Engineering Profession has been content with empirical assumption in place of exact analysis may be attributed to the fact that analytical method of investigation of internal stress has been encumbered by a mass of formulas, perplexing even to the expert mathematician. It was so encumbered because the treatment consisted in a comparison of stresses acting on planes in various directions involving the transformation of quadratic functions of three variables through expressions which contain such a multiplicity of symbols and such a tedious array of direction cosines that even the expert mathematician dislikes to use them. The difficulty lies in the unsuitability of Cartesian co-ordinates for expressing relations which are dependent on the principle of the parallelogram of forces and not in the complexity of the relations themselves, which may be solved geometrically by the systematized application of the parallelogram law. Graphical statics of states of internal stress at a point have been developed by Eddy,* but he made no progress toward stress analysis of beams, plates, and the like with which the designer is called on to deal.

This was because the generalized mathematical theory deals with solids of infinite extent with scant consideration to narrow limitations of boundary of the structural member in which the intensity of the stresses are determined by the manner of support, the variations of section, and the position and direction of the applied forces. These latter conditions are embodied in the simple summation equations of the theory of flexure from which may be determined the exact position of the neutral plane and its departure in the simple beam from the center of gravity of section except at points of maximum moment, where the vertical shear passes through zero. Notwithstanding the complications of analytical methods engineers should not lose sight of the fundamental truths which have been developed thereby and disregard them as completely as has been done by the Joint Committee in its rules of design.

Mathematical theory shows that the sharpest curvature is along the principal planes where the twisting shear passes through zero. Now these planes for the cantilever and the dish-shaped area about the diagonal center of the span are the diagonals of the panel, and the greatest bending moment cannot be calculated as the Joint Committee would do about a median line and parallel column lines.

* "Researches in Graphical Statics", 1878.

In the Committee's analysis, three of eight of the planes of symmetry and zero shear in an interior panel uniformly loaded are considered, as follows: It considers a median line and two column lines and disregards three corresponding planes at right angles which, if considered with the first three, divide the square panel into quarters representing that load area that is transferred to each respective column and an applied bending moment of $\frac{W}{4}$ multiplied by the lever of one-quarter of the diagonal span less one-half the effective diameter of the cap. If the columns were mere points the applied bending moment would be $+\frac{W\,l}{16}$, in which l is the diagonal span in each direction at right angles, and this applied bending moment is the greatest in the cantilever and suspended span. From the triangular areas formed by the diagonal planes of zero shear the twist about the same axes is $-\frac{W\,l}{48}$ to be added algebraically to $+\frac{W\,l}{16}$.

In the saddle-back area midway between columns somewhat sharper curvature occurs than about the diagonal center of the panel with certain arrangements of metal and certain ratios of cap diameter to span length. This was the case with the old four-way construction and the two-way construction commonly used to-day. The diagonal mushroom system by its different arrangement of metal decreases the deflection by approximately one-half at the diagonal center of the panel and a larger amount in the center of the saddle-back area.

The unsatisfactory character of attempted mathematical plate solutions noted by Kelvin may be accounted for by the attempt of the mathematician to develop a generalized formula for the equilibrium of a square or rectangular prismatic element. In undertaking to treat such an element the twisting and bending shears are intermingled in such a complicated way on all four of the vertical faces of the prism that the problem involves greater complications than that of the general one of three bodies in space. This difficulty may be obviated by selecting the elementary prism of such a form that two of its vertical faces shall be (non-parallel) planes of zero shear; that the tangential vertical forces on the other two faces made parallel to OB, where the curvature changes sign, will be a uniform distribution of vertical shear force which gives rise to true bending moment, and acting tangentially in a horizontal direction on these parallel faces we have the plus and minus shearing stresses which follow the linear law in increasing in magnitude from the center of depth to a maximum at the outer fiber. Such a distribution of shears is represented by Fig. 60, in which the axis of Y is normal to OB (Fig. 59), and, in this particular instance, parallel to AC. The faces, $e\ h\ h'\ e'$ and $g\ f\ f'\ g'$, are planes of zero shear and the planes, $e\ f\ e'\ f'$ and $h\ g\ h'\ g'$, are parallel to OB. The reason that such a distribution of shearing forces causes twist lies in the fact that the magnitudes of the twisting shears, plus and minus H, increase in consecutive parallel planes from the

center at C to O where they change sign and decrease along parallel planes from O to A. The twist involved supports the weight of the load in its outward distribution through the material as shear from C to OB and its reverse shift from OB in its travel to the column, A. These twisting shears are responsible for the circumferential squeeze at the periphery of the dish-shaped areas and for the stretch of the entire plate in the saddle-shaped areas. They account for the relatively high intensity of the circumferential stresses at the periphery of the dished areas whereas the circumferential stresses at the center of such areas are largely brought about by the fluid state of stress arising from the presence of equal principal stresses of like kind at right angles.

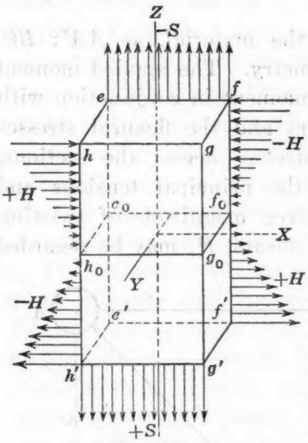

FIG. 60.

After developing the correct theory of the transference of the weight of the load by shear in the most direct manner toward the support in the plate, as demonstrated by the buckling of thin plates and the theory of least work, the writer overlooked the obvious fact in his first efforts that the support of the suspended span is the edge of the cantilever and, in turn, the support of the cantilever portion of the plate is the column, so that in traveling thus wise, the vertical shear is distributed uniformly along the planes where the curvature passes through zero across the anticlastic areas, and the vertical shears in like manner are distributed uniformly by the twist in all parallel planes so that the principal stresses at mid-depth of the plate are readily determined in a dual manner by the intersection of the twisting shears arising from the torsion of the vertical element. Again, this torsion causes the circumferential squeeze which may be demonstrated readily by a model of the circular plate.

As far as the writer is aware this analysis,* is the first successful effort to determine the direction of the principal stresses and the precise distribution of the shears and the curves of displacement and equi-potential in uniform elastic homogeneous plates bent in any manner. The state of stress in the concrete where uniform mats of crossed rods cover the necessary areas is almost identical with the state of stress in a homogeneous plate in plan, but in the principal vertical planes the equilibrium of principal stresses and shears takes place as developed in† beam action.

In conclusion, the writer is of the opinion that exact theory should be of interest to the designer because it gives a satisfactory insight into the mechanics of resistance which building codes tend to obscure and, moreover, an understanding of exact theory should permit the simplification of the rules of design to the specification of proper working stresses which can be verified within 5 or 10% by scientific test instead of differing therefrom by 400 per cent.

* "Elasticity and Strength of Materials", Section III.
† *Loc. cit.*, Section II.

To correlate the stress analysis of Fig. 59 with steel stress for the concentrated central load (for simplicity assuming the column diameter to be negligibly small), the travel of the center of gravity of each eighth part of the load may be traced through its octant to the support. (See Fig. 61.) By the twisting couple, plus or minus H, the center of gravity of $\frac{W}{8}$ in the octant, ABC, moves to m at the center of OB. Hence, the total twisting couple is $\frac{W}{8} \times O\,m = \frac{W}{8} \times \frac{l}{8} = \frac{W\,l}{64}$ positive in the convex dished portion and $\frac{W\,l}{64}$ negative in the concave portion, and the numerical sum of these statical moments resists deformation of the material, as AA'; BC, and CD are planes of zero vertical shear by symmetry. The applied moment between A and C is reduced by their sum as this moment in conjunction with that at right angles is the cause of these shears and the flexural stresses incident to them through horizontal moment stresses across the sections, CB, CD, AB, and AD. As in pure torsion the principal tensions and compressions are only one-half as great for a given magnitude of twisting moment as in simple flexure, the plus for minus shears, H, may be regarded

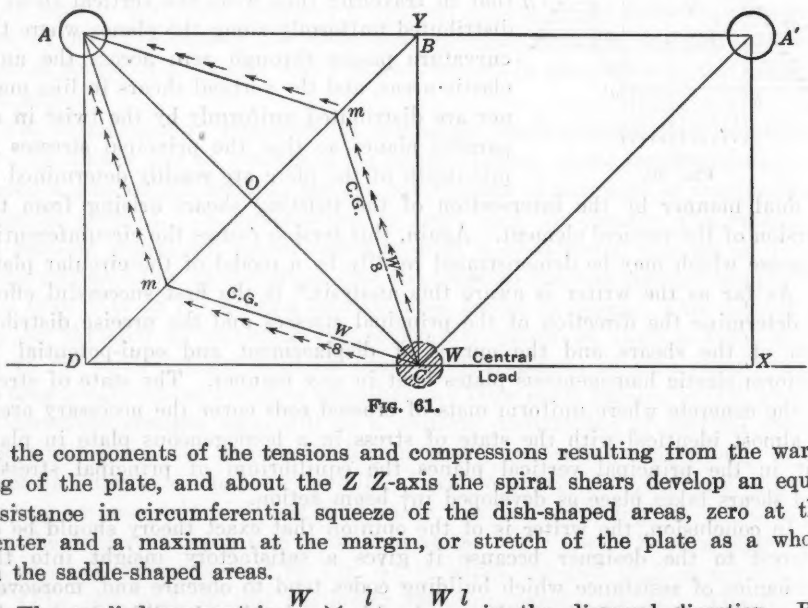

FIG. 61.

as the components of the tensions and compressions resulting from the warping of the plate, and about the $Z\,Z$-axis the spiral shears develop an equal resistance in circumferential squeeze of the dish-shaped areas, zero at the center and a maximum at the margin, or stretch of the plate as a whole in the saddle-shaped areas.

The applied moment is $\frac{W}{4} \times \frac{l}{2} = \frac{W\,l}{8}$ in the diagonal direction. Reducing this by $\frac{W\,l}{32}$ for the plus or minus twist about $A\,C$ and a like amount for the twist about ZZ, $\frac{W\,l}{16}$ remains to be resisted by the steel

stress parallel to AC. This constitutes a static justification of Eddy's proposal to treat the steel mat as a sheet of steel in its effective resistance. The result may be compared with that of the structural steel methods of the Joint Committee.

The Committee's applied moment (treating the diameter of the column as negligible) would be:

$$\frac{W}{2} \times 0.7071 = 0.7071 \frac{W l}{4} = 0.177 \ W l$$

or it would require 2.828 times the relative steel stress computed previously.

It is obviously impossible to twist or warp a plate without developing resistance to such distortion capable of opposing applied moments, as the magnitude of the total applied moment is independent of the kind of resisting structure. Equilibrium results from a balance of the sum of internal resistance of all kinds, twisting and bending combined with stretching or squeezing of the plate as a whole in the various sections. All forms of resistance must be given due weight if theoretical computation and experimental measurements are to agree. Another element of resistance found in the continuous beam and slab as well, is the so-called arch or dome action which arises from the interference of shear distortion at the point of inflection. The reduction in steel stress from this cause is of like amount numerically at mid-span and support.

The prevailing opinion in 1906 regarding concrete construction was expressed by John S. Sewell, M. Am. Soc. C. E.* in the statement that no economic system of floor construction could be built without rolled steel beams or concrete T-beams. The writer alone took exception to this and presented a design of mat formation of the reinforcement which has since been used the world over. Engineers commonly declared at that time that the construction could not be calculated, it was crazy to put it up, until after it had been embodied successfully in various structures.

Its general adoption is a vindication of the fundamental principle that economic lines of strength are essentially graceful lines, the flat slab in building construction being neater than the beam and slab types which it has replaced for heavy load structures. It is beginning to be adopted extensively in bridge construction. The new Mendota-Snelling Bridge, 4 200 ft. long, consisting of thirteen 300-ft. arch spans, with a deck 61 ft. wide, has a continuous monolithic plate for its floor. The compression members for its 300-ft. arches are hooped and rodded longitudinally as well. It presents a neatness of appearance seldom approached, at a contract cost of one-half that of arch viaducts for which the formulas for beam construction members, embodied in the Joint Committee's rules, have been followed. The arrangement of metal in these flat slabs differs from that of the older forms in that reinforcement is provided at 45° to the column lines at mid-span, opposing the stretch of the plate in the area of anticlastic curvature.

As the engineer develops his knowledge of exact theory so that the structures follow the graceful lines of most economic strength, the propaganda

* *Transactions*, Am. Soc. C. E., Vol. LVI (1906), p. 252.

that he should be supplanted in charge of city bridges by the architect for the development of the city beautiful will fall of its own weight. To achieve this desideratum an approach to exact theory much closer than the 200 or 300% limit of the Joint Committee's rules for shear distortion must be attained.

WALTER H. WHEELER,* M. AM. SOC. C. E. (by letter).†—The report of the Joint Committee represents the results of a long study and investigation by a group of engineers who deserve the whole-hearted thanks of the profession. In many ways this report marks progress in the art of constructing reinforced concrete, and it is to be hoped that it will aid in improving the quality of concrete and placing the art of making it on a more exact and scientific basis. One great step has been taken in giving credit in the design of columns for stronger concrete, which allows the engineer some latitude in design.

There are certain features of the specifications with which the writer cannot agree, notably Section 22, covering standard sizes for reinforcing bars. It seems unfortunate that a group of steel manufacturers should decide on a certain schedule of sizes and shapes to be rolled and that those sizes should be adopted without obtaining the opinion of the entire Engineering Profession on such a drastic change. The writer is in full accord with the purpose of simplified practice when it does not result in injury to any of the users of the product, or to the products involved. The Division of Simplified Practice of the U. S. Department of Commerce states clearly that the number of users affected or the value of their output is not the controlling factor. If any part of the industry is damaged that is sufficient cause for giving that part consideration. Simplified practice means—eliminate waste not make waste.

The schedule adopted by the Division of Simplified Practice contains one more size for round bars, namely, $\frac{1}{4}$ in., than the schedule set forth in the specifications of the Joint Committee. The writer wishes to enter his objections to this schedule of sizes for the following reasons:

1.—In the design of solid concrete slab construction, particularly light slabs, good practice requires that a maximum spacing center to center of main reinforcing bars will not exceed $1\frac{1}{2}$ times the effective depth of the concrete slab.

2.—Many slabs are constructed for which $\frac{3}{8}$-in. round bars are too large to permit of this maximum spacing without using excess steel.

3.—The mill extra for $\frac{1}{4}$-in. round bars over and above the base price is $1.00 per 100 lb. The mill extra on $\frac{5}{16}$-in. round bars is $0.70 per 100 lb., therefore, if $\frac{1}{4}$-in. round bars are used where $\frac{5}{16}$-in. round bars should be approximately 10% is added to the cost of the reinforcement required.

4.—If $\frac{3}{8}$-in. round bars are used and extra bars added to provide for maximum spacing, the weight of the steel may be increased approximately 50% above that required if $\frac{5}{16}$-in. round bars were available.

5.—Bars $\frac{3}{8}$-in. square, or $\frac{7}{16}$-in. round, are another size for which there is a large use in reinforced concrete construction. The mill extra for $\frac{7}{16}$-in.

* Designing and Cons. Engr., Minneapolis, Minn.

† Received by the Secretary, December 6, 1924.

round bars is $0.10 less per 100 lb. than for ⅜-in. square bars of approximately the same sectional area.

6.—The sectional area of 1⅛-in. round bars is practically the same as 1-in. square bars and the sectional area of 1¼-in. round bars is practically the same as 1⅛-in. square bars.

7.—Round bars in reinforced concrete work are more satisfactory to handle, can be embedded in the concrete work with more satisfactory results, and where the same depths from the concrete surface are maintained, the round bar has more fire-proofing than the square bar, unless the square bar is placed cornerwise, which cannot readily be done.

Based on these considerations, the writer wishes to urge the Joint Committee to revise this schedule, as follows:

¼-in. round bars	0.049 sq. in.	¾-in. round bars	0.441 sq. in.
5/16-in. round bars	0.077 sq. in.	⅞-in. round bars	0.601 sq. in.
⅜-in. round bars	0.110 sq. in.	1-in. round bars	0.785 sq. in.
7/16-in. round bars	0.150 sq. in.	1⅛-in. round bars	0.994 sq. in.
½-in. round bars	0.196 sq. in.	1¼-in. round bars	1.227 sq. in.
9/16-in. round bars	0.249 sq. in.	1⅜-in. round bars	1.485 sq. in.
⅝-in. round bars	0.306 sq. in.		

Section 70.—In the writer's practice he has never been able to discover any advantage in casting columns several hours before the slabs and beams for stories of ordinary height, provided the concrete does not contain an excessive quantity of water or unless the columns contain a different concrete mixture from that in slabs and beams. He prefers to pour the columns just before the pouring of the slabs and beams so that the concrete in the columns will have only about 30 min. to settle in advance of the pouring of the slabs and beams. He prefers not to have a construction joint in columns.

Section 25.—Why does the Joint Committee specify cold drawn wire, which covers wire for spiral reinforcement? Why not also rolled rods, either of structural or intermediate grade?

Section 74.—From the writer's observation, expansion joints in buildings had better be omitted. In any heated building, the temperature variation is small throughout the year. He has frequently observed that expansion joints in buildings are a source of trouble. Cracking in buildings, due to temperature and shrinkage, is usually well distributed and is little affected by expansion joints 200 ft. apart.

Referring to Chapter XI of the report, the writer likes to believe that engineers base their conclusions on facts. The first thing he would expect to find in a report which recommends such wide departures from common practice would be an imposing array of facts, or at least a complete list of references to all published test data. Instead the conclusions of the Joint Committee are given, unsupported by a single test or test reference. Certainly, if the Committee expects its recommendations on design to be adopted as a standard by the profession, the authority for its conclusions should be given.

Table 51 gives a comparison of the design of columns proposed by the Joint Committee with that contained in the building ordinance of Minneapolis, Minn., which comparison discloses some very remarkable values.

TABLE 51.—COLUMN COMPARISONS, BASED ON CONCRETE STRENGTH OF 2 200 POUNDS PER SQUARE INCH.

Column design.	JOINT COMMITTEE CODE.			MINNEAPOLIS BUILDING ORDINANCE: Spiral cols., 800 lb. sq. in. on core; " " 10 000 " " " " vertical steel; Considere ratio on spirals		
	Total load, in pounds.	Load per square inch core, in pounds.	Load per square inch total area, in pounds.	Total load, in pounds.	Load per square inch core, in pounds.	Load per square inch total area, in pounds.
Size, 12 by 12 in.; Verticals, 2 sq. in.; Ties, ¼ in., round, 8 in. c. c.	75 680	1 180	525	50 000	690	347
Size, 12 by 12 in.; Verticals, 2 sq. in. 4%; Spiral, ¼ in., round, 1½ in. c. c.	66 690	1 325	463	95 000	1 670	660
Size, 14 by 14 in.; Verticals, 3 sq. in.; Ties, ¼ in., round 8 in. c. c.	104 720	1 047	535	76 125	690	388
Size, 14 by 14 in.; Verticals, 3 sq. in. 3½%; Spirals, ¼ in., round, 1⅛ in. c.c.	96 638	1 230	494	138 400	1 600	705
Size, 16 by 16 in; Verticals, 4 sq. in.; Ties, ¼ in., round, 8 in. c. c.	137 280	955	537	106 000	680	414
Size, 16 by 16 in.; Verticals, 4 sq. in. 3½%; Spirals, ¼ in., round, 1¾ in. c. c.	139 373	1 235	544	175 500	1 425	685
Size, 18 by 18 in.; Verticals, 6 sq. in; Ties, ¼ in., round, 8 in. c. c.	179 520	915	555	147 000	700	454
Size, 18 by 18 in.; Verticals, 6 sq. in. 3.9%; Spirals, ¼ in. round, 1⅛ in. c. c.	205 300	1 335	633	242 300	1 470	750
Size, 24 by 24 in.; Verticals, 11 sq. in.; Ties, ¼ in., round, 8 in. c. c.	321 000	803	557	287 000	684	500
Size, 24 by 24 in.; Verticals, 11 sq. in., 3½%; Spirals, 5/16 in., round, 1¾ in. c. c.	396 000	1 224	688	472 000	1 480	818
Size, 30 by 30 in.; Verticals, 18 sq. in.; Ties, ¼ in., round, 8 in. c. c.	506 880	750	563	476 000	678	528
Size, 30 by 30 in.; Verticals, 18 sq. in., 3.4%; Spirals, ⅜ in., round, 2 in c. c.	655 000	1 195	728	784 000	1 455	870

Test data with which the writer is familiar do not tend to inspire confidence in rodded and tied columns for high beaming values, and he has always been under the impression that columns reinforced with a combination of vertical steel and spiral hooping in proper proportions and properly distributed are much more reliable under a high unit compression stress than tied columns. The records of failures in reinforced concrete buildings support this belief.

An examination of Table 51 discloses the fact that the Joint Committee recommends a unit stress based on the core area of 1 180 lb. per sq. in. for

a 12 by 12-in. tied column and a unit stress on the core area of a spiral hooped column with 4% of vertical steel, equal to 1 325 lb. per sq. in. If these two columns are compared on the basis of total sectional area it is found that the unit stress of the tied column is 525 lb. per sq. in., and that of the column reinforced with verticals and spirals is 463 lb. per sq. in., a most astonishing comparison. Is there any basis on which so much more credit is to be given to the fire-proofing of a tied column over that of a spiral column? If the fire-proofing is destroyed by fire as it has been, are engineers to prefer a tied column with a unit compression stress of 1 180 lb. per sq. in. to a column with vertical reinforcement and spiral hooping with a unit stress of 1 325 lb. per sq. in.?

Again, referring to Table 51, it will be noted that the unit stress on the core area of tied columns, according to the Joint Committee's formula, reduces from 1 180 lb. per sq. in. for a 12 by 12-in. column, to 750 lb. per sq. in. for a 30 by 30-in. column. The writer prefers to reverse the order and use higher unit stresses on large columns if there is to be any difference. Again, by an examination of Table 51, it is found that according to the Joint Committee's formulas, a tied column is stronger than a column of equal size having 3½% vertical steel and 0.9% spiral reinforcement for all columns with an outside diameter of less than 16 in.

It seems strange that the Joint Committee should introduce two items, p and A, into Formula (42)* for spirally hooped columns when one, A_s, the sectional area of vertical steel, is the product of p and A and is known. At this point attention should be called to the fact that the unit stress on the core area of the 14-in. round column calculated according to the Minneapolis Building Ordinance, which is based on the Considére ratio for spirals, is 1 600 lb. per sq. in. This is due to the fact that the spiral hooping is in excess of the "one-fourth of the vertical steel" required by the report. Wire spirals of ¼ in., would have a spacing of 2¼ in., or more than the one-sixth of the core diameter allowed, and this spacing has been reduced which increases the amount of spiral. If allowance is made for the excess spiral, the unit stress on the core area of the column designed according to Minneapolis Code becomes 1 450 lb. per sq. in., or substantially the same as that for all other sizes considered, except the 12-in. column, which also has an excess of spiral when based on the Joint Committee's requirement of "one-fourth of the vertical steel".

In discussion of previous reports of the Joint Committee, the fact has been brought out by Edward Godfrey, M. Am. Soc. C. E., and others, and tests have been mentioned, to prove that very little strength is added to a column by vertical rods and light ties spaced far apart. Apparently, the Joint Committee has either overlooked these tests, or has ignored them.

The column tests made by several different authorities and referred to† by F. R. McMillan, M. Am. Soc. C. E., a member of the Committee, indicate that a column made of concrete with an ultimate strength of 2 600 lb. per sq. in. and reinforced with 4.6% of vertical steel and 1% of spirals, develops,

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1203.

† *Proceedings*, Am. Concrete Inst., 1921.

after 45 to 60 days, an ultimate strength of 5 060 lb. per sq. in. on the core area and a yield point of 4 960 lb. It is fair to assume that this concrete had a strength at least 25% greater than it had at 28 days. According to the rules of the Joint Committee, which would increase the spiral on this column to 1.15%, the allowable load is 1 365 lb. per sq. in., or 27½% of the yield point.

According to tests by Turner,* columns having 3.12 to 5.17% of vertical steel and 2.01 to 2.53% of hooping, developed ultimate strengths of 7 350 lb. to 8 850 lb. per sq. in., respectively, on the core area. The concrete in these columns was the equivalent of a 1 : 2 : 4 mix. The reported yield point was only slightly less than the ultimate strength. On the basis of the Joint Committee report, the allowable working stress on these columns would not exceed about 1 400 lb. per sq. in., or 19% of the ultimate strength.

It would seem from the foregoing that the work of the Joint Committee is going forward in the art of making better concrete and backward in the working stresses that may be allowed upon it.

There are hundreds of reinforced concrete buildings designed on the basis of the Considére ratio for spirals, 12 000 lb. per sq. in. on vertical steel and 1 000 lb. or more on the concrete core area. Some of these buildings have been through severe fires; others have been grossly overloaded. The quality of the concrete in some is poor and, in most of them, it is not equal to the average that should be had for 1 : 2 : 4 concrete under the Joint Committee's specifications. To the writer's knowledge there is no indication of distress in any such columns or that there has ever been any. Mr. Godfrey has referred† to the fact that there have been a number of failures of tied columns; others have noted this fact.

Where then does the Joint Committee find the evidence to justify its column formulas? Why is it considered necessary to demand a factor of safety of 4 to 5 on spirally hooped and vertically reinforced columns based on the strength of concrete at the age of 28 days when the Committee knows that the strength will probably increase 50% within 4 to 6 months and when it also knows that these columns will not receive their full load in less than 3 months after they are poured, for any average building in which high column stresses are required? Why are tied columns given such a preference? Why require a factor of safety of 5 or 6 on reinforced concrete which grows stronger with age and a factor of safety of 2 to 3 on steel columns which grow weaker with age?

According to the rules for the design of T-beams, if a building has a girder span of 40 ft. and a clear slab span of 16 ft., and the slab is 6½ in. thick, the part of the slab considered as belonging to the T-section of the beam would be 52 in. wide on each side of the beam, but if the span of the beam is reduced to 30 ft. and the beam is 16 in. wide, the T-section could be only 37 in. wide on each side of the beam, and if the span reduces to 20 ft. and the beam is 12 in. wide, the T-section reduces to 24 in. wide on each side

* "Concrete Steel Construction", Part I, by Eddy and Turner.

† *Transactions*, Am. Concrete Inst., 1921.

of the beam, or less than one-half the width for a 40-ft. span. The writer is unable to find any basis in tests or elsewhere for such a rule.

In the design of flat slabs, the writer is certain that the Joint Committee is far from the truth in the solution of this problem, and he desires to offer certain facts which he has observed and which others have noticed,* in evidence. Moderate sags in flat slab floors which have gradually manifested themselves even in floors which are not loaded, are not evidence of weakness any more than are similar sags in wood joist floors. There is scarcely a wood joist floor to be found that has no sags. The sags in concrete slabs referred to, are the result of shrinkage in the concrete and of re-adjustments in the concrete matrix, resulting from various causes, but usually noticeable in concrete which has been mixed and placed wet or over-stressed when green by too early removal of forms or settlement of forms. This same condition seems to be manifested in columns by a slight shortening, as observed by Messrs. F. R. McMillan and M. B. Lagaard.†

There has been much discussion of flat slab theory and many tests have been published. The writer is disappointed that the Joint Committee has been unable to bring about a closer reconciliation between theory and fact than is shown by its report.

Take two constructed buildings, for example, both of which are ten years old and in neither of which is there any evidence of cracking, settlements, or weaknesses of any kind. These buildings have been carrying their normal floor loads for ten years.

The first building has maximum panels of 15 ft. $3\frac{1}{2}$ in. by 18 ft. The concrete floor slab is $6\frac{1}{2}$ in. thick, and the columns are 16 in. square. The column capitals have a flare of 3 in. and a depth of 4 in. There are no depressions; part of the building is only two panels wide and there is only one-half as much steel in the slab as is required by the Joint Committee's formula. The concrete was of average quality. The building is partitioned for offices above the first floor which is open so that no support to floors is given by partitions.

This same building designed according to Joint Committee rules would require a total thickness of slab and drop panel of 11 in., with a drop panel 6 ft. by 5 ft. 1 in. by 5 in. and a 6-in. slab between the drop panels. It would require 100% more steel, calculated on the basis of 18 000 lb. per sq. in. for positive moment, than was put into the slab when built.

The other building is two panels wide, one panel being 6 ft. longer than the other. The first floor is open, but the upper floors are partitioned. The panels are 22 ft. by 16 ft., the slab is $7\frac{1}{2}$ in. thick; the columns are 16 in. square, and the capitals have flare of 3 by 4 in.

This building designed according to Joint Committee rules would require a total thickness of $13\frac{1}{2}$ in. for slab and drop panel; drop panels, 7 ft. 4 in. by 5 ft. 4 in. by 6 in. thick; $8\frac{1}{2}$-in. slab between panels; and about 100% more reinforcement for positive moment than was used in the slab when constructed.

* F. R. McMillan, M. Am. Soc. C. E., in *Proceedings*, Am. Concrete Inst., 1921.

† *Proceedings*, Am. Concrete Inst., 1921.

These buildings are two of many similarly constructed that could be mentioned and that show similar evidence of being designed to meet the requirements.

If the writer correctly understands the formulas of the Joint Committee for four-way slabs without depressions, the compression on the concrete in the slab last described, due to negative moment at normal load, would be 1 500 lb. per sq. in. in the outer fiber. The compression due to positive moment in the outer fiber would be 1 190 lb. per sq. in., and the tension in the steel at mid-span would be 39 000 lb. per sq. in. The steel used had an ultimate strength of 55 000 to 60 000 lb. per sq. in. and a yield point of about 32 000 lb. per sq. in. No compression tests were made on the concrete. The mix was 1 : 2 : 4, placed moderately wet. The coarse aggregate was good, but the fine aggregate was rather poor, so that it is not likely that the concrete would show more than 2 000 lb. per sq. in. in 28 days. If the actual stresses are equal to those determined by the Joint Committee's formulas, there must be some miraculous qualities in a flat slab which not only make it stand up, but prevent it from showing any deflections with such stresses.

Another building with which the writer is familiar is designed to carry a working live load of 300 lb. per sq. ft. on the same basis as the two buildings hereinbefore described, except that the column capitals have a diameter equal to 0.18 of the span, but no drop panels. This building is about 14 years old, and carried for several months of each successive year a working live load of 425 lb. per ft. over almost the full area of the entire floors of the building. The grade of the steel is the same as that previously described, of low ultimate strength and yield. The concrete is the same mix, but better aggregates were used. After all these years of such severe usage the floors hold their shape and are apparently as good as when the building was first constructed. The writer knows of a great many buildings and other structures which were similarly designed and have a similar good record of performance. It is certain they would not show this record if the formulas of the Joint Committee are correct or approximately correct. The Committee attempts to account for the great strength of flat slabs of some designs, as compared with their calculated strength, on the assumption that it is the tensile strength of the concrete. The writer has no faith in the truth of this assumption. He believes that the trouble with the Joint Committee's formula is the result of failure to give the effects of shear and combined twisting and bending proper weight. Some credit has been given to these agencies and it is enough for slabs designed on the beam-strip theory. The writer sincerely hopes that the Joint Committee will make another attempt to solve this problem. Until such time the writer prefers to rely on the theory of flat slabs as devised by Dr. H. T. Eddy and which agrees closely with results actually attained in the field, rather than to accept a theory which does not agree with field results and which must be explained by giving credit to the tensile strength of concrete, although in Section 103, Item (*e*) of the report,* it is stated that tensile strength is assumed to be neglected.

* *Proceedings*, Am. Soc. C. E., October, 1924. Papers and Discussions, p. 1187.

The impression seems to be quite general in the Engineering and Architectural Professions that flat slab construction is not applicable to light buildings. The writer has found the reverse to be true. Properly designed flat slabs are ideal construction for light load buildings, but, unfortunately, many city building codes are drawn so as to preclude their use. In applying flat slab, or any other reinforced concrete construction to light load buildings, it is necessary to consider the construction loads which may come on the floors. Reinforced concrete slabs which are overstrained during construction, or while green, cannot be relied on to hold their shape, although they may be satisfactory as to strength. There is a tendency among contractors and workmen to look on a reinforced concrete floor much as they do a pavement in the street and to load it accordingly. It is entirely practical to avoid careless overloading of floor slabs during construction, but slabs should be designed for loads which provide for practical construction methods and this cannot be done if such loads as 40 and 50 lb. per sq. ft. are the basis of slab design. On the other hand, it is not necessary to provide for construction loads in the design of columns in the ordinary building. In the writer's experience, part of the prejudice against flat slab construction for light load buildings and buildings which are to be partitioned, can be traced to this damage during construction, and it is naturally more manifest in such slabs than in other types of construction where slabs are thicker in proportion to span. However, the writer has observed this same difficulty with other forms, such as one-way slabs and joist slabs. It is a matter which can be, and should be, covered in building codes, and for which engineers should provide in their designs regardless of building code requirements.

One serious defect in building codes is the failure to recognize and give credit in the design for the scientific handling and placing of concrete. The report of the Joint Committee bases working stresses upon strength of concrete. It is very conservative in the stresses recommended. Concrete which is the equivalent of a 1 : 2 : 4 or 1 : 2 : 3.5 mixture can be depended upon to develop consistently 2 500 lb. per sq. in. in compression at the age of 28 days, provided the aggregates are carefully graded and accurately measured, the mixing is thorough, and the water ratio does not exceed 7 gal. per sack of cement. On the other hand, this concrete might drop well below 2 000 lb. per sq. in. if the aggregates are poorly graded, carelessly measured, and an excess of water is used in mixing. The writer has proved this proposition on his own work in the field. Therefore, he contends that rules of design in concrete should take into consideration the specifications under which the work is to be executed and the supervision to be given in the field. The writer has found that it is both possible and practicable to produce 1 : 2 : 4 concrete in the field, which at the end of 28 days will test regularly and uniformly 2 500 to 2 800 lb. per sq. in. in compression. If the specifications are drawn so that such results can be depended upon and careful control and supervision is to be given to the work in the field, it should be possible to capitalize such results and use proportionately higher design stresses. A provision in building codes

which would allow engineers to capitalize the results of their skill, would save millions of dollars every year in the construction of buildings.

The writer is a firm believer in the principle that successful engineering means the obtaining of maximum economy in construction without sacrificing strength, durability, utility, or appearance, and that such engineering is a National asset. He believes just as firmly, however, that wasteful engineering designs are not the result of successful engineering and are a National liability. We must get the truth and the whole truth as far as it is humanly possible about the design and construction of reinforced concrete. The writer wishes again to commend the Joint Committee and to urge it to further effort.

CHANDLER DAVIS,* M. AM. SOC. C. E. (by letter).†—The Standard Specifications for Concrete and Reinforced Concrete as submitted by the Joint Committee are required and should be adopted, for a standard practice is needed in the United States in view of the numerous failures on record.

The writer has heard engineers voice the opinion that water-tight forms are unnecessary, in fact, one engineer told him that he intentionally leaves openings in his forms; further, that it was a useless expense to place the reinforcing steel accurately and secure it against displacement. The discussion was in reference to the construction of columns which stood in tidal waters, the lower 3 ft. of which were submerged twice a day. The writer was told that no particular precaution was taken to prevent the water of the concrete from escaping. That such an occurrence would result in removing cementing material from the concrete did not seem to matter very much. It was also stated that a ½-in. covering to the steel reinforcement would be sufficient protection against the action of the water, although the current of the river in which the structure stood was very swift.

M. M. O'Shaughnessy, M. Am. Soc. C. E., in his paper entitled "Ocean Beach Esplanade, San Francisco, California",‡ describes a structure which was commenced in 1915 and has successfully withstood the action of the sea and the climatic conditions. In this structure the minimum covering to the steel reinforcement is 3 in., the maximum, 5 in. He lays stress on the necessity of making concrete impermeable and of properly placing, securing, and covering the reinforcement. All this seems elementary, yet the writer has frequently heard it said that such care simply increases the cost of the construction needlessly; he believes that these points should be incorporated in some standard specifications such as those proposed by the Joint Committee and should be adopted by the Society. The writer further believes that a clause should be inserted in these standard specifications describing the reinforcement which may be placed over the opening of the column forms, before placing the concrete. He has been told seriously that it is "good American practice" to choke the forms with the reinforcement of the longitudinal and cross-girders and the floor steel and the steel in the brackets, even if there should be two or more layers of such meshes. This arrangement of steel may

* Cons. Engr., New York, N. Y.

† Received by the Secretary, January 7, 1925.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 492.

reduce the openings through which the concrete is poured to 2 in., or less. It will make proper rodding difficult, if not impossible. Tamping is out of the question. The tapping or hammering of the forms opposite the freshly deposited concrete is depended upon to compact the concrete. It is needless to point out that such a process is useless as far as the concrete in the center of the column is concerned.

Standard specifications for concrete and reinforced concrete are needed and should be approved and adopted by the Society. Once such specifications have been adopted, however, steps should be taken to have them incorporated in the building codes of the various States and cities.

The Joint Committee must be congratulated on the thorough manner in which the subject of concrete, both plain and reinforced, has been studied, and it is to be hoped that the conclusions reached will be thoughtfully discussed and that any criticism offered will be constructive.

R. W. GAUSMANN,* M. AM. SOC. C. E. (by letter.)†—The report on Field Tests of Concrete, by W. A. Slater, M. Am. Soc. C. E., and Mr. Stanton Walker,‡ should prove of great interest to all members of the profession who are working with concrete. The authors have presented valuable data, usually very difficult to collect in the rush and hurry of a construction job. They have proved that even when using ordinary field methods, concrete having a reasonably consistent strength is produced. Although the data presented in this report are generally very complete, the writer differs in certain conclusions which have been drawn from these data and also feels that other conclusions might have been reached.

The purpose of the report was to demonstrate the practicability of the Tables of Proportions.§ It would seem logical, therefore, to make some comparison between the mixes actually used and those which might have been taken from the tables. The authors have not done this. Using data given in the report, the writer has attempted to make this comparison and finds that had the mixes been determined from the tables a much larger quantity of cement would have been used. Such being the case it would seem that Conclusion 22,‖ "that it is possible to meet requirements based on the Tables of Proportions as contained in the report of the Joint Committee", has not been proved.

These experiments were conducted on two different jobs by the same personnel, and it would seem that some comparison should have been made between the two. As the authors did not do this, the writer, after a careful examination of the data in the report, has arrived at the following conclusion:

The leaner mixes were practically identical on both jobs, yet the average strength of these two mixes differed by 1 000 lb., or one was 46% stronger at 28 days than the other. Without knowing all the conditions, it is difficult

* Div. Engr., New York City Board of Water Supply, Grand Gorge, N. Y.

† Received by the Secretary, February 24, 1925.

‡ *Proceedings*, Am. Soc. C. E., January, 1925, Papers and Discussions, p. 3.

§ See Appendix XVI, "Standard Specifications for Concrete and Reinforced Concrete," *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, p. 1153.

‖ *Proceedings*, Am. Soc. C. E., January, 1925, Papers and Discussions, p. 6.

to assign a reason for this great discrepancy in strength. Fig. 13* would seem to indicate that the discrepancy was due to the difference in slump, in fact it happens to give an exact mathematical check. Fig. 12†, however, refutes this theory completely. For instance, at Camden, N. J., a strength of 2 200 lb. is obtained with slumps varying from 4 to 10 in., whereas at Newark Bay a strength of 3 200 lb. was obtained with slumps of from 1½ to 7 in. The discrepancy in the fineness modulus, or a difference in the grading of the aggregate which is not disclosed by the fineness modulus, may account for some of this difference in strength, also different brands of cement may have accounted for this discrepancy.

Before this report is accepted and the Tables of Proportions finally adopted, it is suggested that more conclusive proof be offered, that the results of the experiments confirm the data in the tables. The authors should also offer some explanation for the difference in strength which was obtained at the two places where experiments were conducted.

EDWARD E. BAUER,‡ ASSOC. M. AM. SOC. C. E. (by letter).§—As a part of the regular work in Civil Engineering at the University of Illinois, members of the Senior Class take a course in what is known as "Plain Concrete", during which tests of cement, aggregates, mortar, and concrete are made. The results of such tests for 1924-25, on mortar and concrete are of interest in connection with the "Report of Field Tests of Concrete", by W. A. Slater, M. Am. Soc. C. E., and Mr. Stanton Walker.‖ Although the tests were not made under field conditions, it is felt that they are fairly representative of what might be achieved on a job.

In addition, a comparison is obtained of the results of different individuals or groups doing the same work under the same conditions. It is interesting to note the similarity of some of the values obtained by the different groups. It must be remembered that the students performing the tests are doing their first work in concrete testing.

ORGANIZATION

The Seniors were divided into four groups working on different days of the week. The smallest group contained eight, and the largest, twenty-three, men. The groups are designated by the letters, *E, F, G,* and *H*. Each student made tests of the cement and sand and, the class acting as a unit made tests on gravel, mortar, and concrete. In order that all students might cover the same ground, each class made the same tests under the same conditions.

LAYOUT OF TESTS

This discussion concerns principally the tests on mortar and concrete. For convenience, the problems have been given the same numbers in this discussion as in the class work.

* *Proceedings*, Am. Soc. C. E., January, 1925, Papers and Discussions, p. 27.

† *Loc. cit.*, p. 26.

‡ Asst. Engr., City of Urbana; Instr., Dept. of Civ. Eng., Univ. of Illinois, Urbana, Ill.

§ Received by the Secretary, March 13, 1925.

‖ *Proceedings*, Am. Soc. C. E., January, 1925, Papers and Discussions, p. 3.

Problem 15.—Compression Test of a Mortar.—The mix was 1: 2 by weight. The water content was varied.

Problem 20.—Effect of Mixing Water on the Strength of Concrete.—The mix was 1: 2: 3.5 by weight, or 1: 1.72: 3.13 by volume. This is approximately a 1: 3.88 mix, in which the 3.88 represents the bulk volume of the combined aggregate. The quantity of mixing water was changed so that the consistency varied from a dry mix to a very wet one.

Problem 21.—Effect of Curing Conditions on Strength.—The mix was the same as that used in Problem 20. The specimens were cured in a moist room, in a tank of water, and in the laboratory air without any moisture.

Problem 22.—Effect of Time of Curing on Strength.—Only one set of tests was made in this problem. The mix was the same as that used in Problem 20. The specimens were removed from the forms at the end of 24 hours and placed in the moist room until about 1 hour before testing. Three specimens were broken on each of 12 days, the age varying between 1 day and 30 days.

Problem 23.—Design of Mixtures.—In this problem a number of items were varied. There were twelve batches. Eight had, as the fine aggregate, the coarse sand of previous problems. In the other four, a fine plastering sand was used. The first eight batches were divided into four mixes with two consistencies for each mix. The last four batches were divided into two mixes and two consistencies for each mix. The last four batches were designed according to Abrams' method to give the same strength as four similar batches using the coarse sand as a fine aggregate.

MATERIALS

Cement.—The cement was part of a shipment received in May, 1924, by the Department of Theoretical and Applied Mechanics for experimental work. It was stored in the laboratory and showed some signs of caking. The results of tests on cement are given in Table 52.

TABLE 52.—RESULT OF TESTS ON CEMENT.

(Results of One Student from Each Group Selected at Random.)

	E.	*F.*	*G.*	*H.*
Soundness	O.K.	O.K.	O.K.	O.K.
Percentage of fineness*	17.0	17.1	15.3	16.6
Initial set	1 hour 2 min.	1 hour 30 min.	1 hour 4 min.	1 hour 40 min.
Final set	Within 10 hours			
Tension, in pounds per square inch: 7 days	213	182	......	163
Tension, in pounds per square inch: 28 days	305	328		295
Specific gravity as received†	3.12	3.03	2.95	3.02

* Machine shaking.

† Test not made on ignited sample.

Fine Aggregate.—Fine aggregate for Problems 15, 20, 21, 22, and part of 23, was a coarse sand brought from Attica, Ind. In part of Problem 23 a fine plastering sand was used, which was brought from Lincoln, Ill. The sieve analyses as determined by the different groups are given in Table 53.

TABLE 53.—SIEVE ANALYSIS.

(Percentage Coarser.)

Size of sieve.	E.	F.	G.	H.	Average.	Plastering sand.
100	98.4	98.4	99.0	94.2	97.5	99.2
48	94.0	94.6	94.8	93.0	94.1	81.8
28	75.2	77.8	74.8	73.8	75.4	29.2
14	52.0	56.6	52.2	52.3	54.3	11.6
8	30.6	34.7	29.8	31.2	31.6	2.4
4	9.4	10.2	8.0	9.3	9.2	0
⅜	0.0	0.0	0.0	0.0	0.0	0
Fineness modulus	3.596	3.723	3.586	3.538	3.62	2.24

The quantity of silt was determined by washing a 500-gram sample until the water was clear, drying the sample, and weighing again. The percentage is based on the original weight.

	E	F*	G	H	Average
Silt test	2.8	0	2.0	0.8	1.9

The colorimetric test in all cases showed no organic matter to be present.

Coarse Aggregate.—The gravel used was brought from Attica, Ind. The sieve analyses made by the students are given in Table 54.

Water.—The mixing water was taken from the University drinking supply.

TABLE 54.—SIEVE ANALYSIS BY STUDENTS OF GRAVEL FROM ATTICA, IND.

(Percentage Coarser.)

Size of sieve.	E	F.	G.	H.	Average.
No. 4	100.0	100.0	100.0	100.0	100.0
No. 3 (¼ in.)	87.0	92.0	88.5	89.3	89.2
⅜ in.	64.4	74.0	67.5	68.8	68.7
¾ in.	22.0	41.2	27.7	31.2	30.5
1 in.	9.9	27.8	13.6	18.2	17.4
1½ in.	0	4.9	0	5.5	2.6
2 in.	0	0	0	0	0
Fineness modulus	6.86	7.20	6.95	7.05	7.02

MANIPULATION

Measuring Materials.—All the materials, including the water, were weighed. The aggregates were bone dry. In calculating the water-cement ratio, 2% of the volume of the combined aggregate was allowed for absorption.

* Apparently an error was made in weighing. Omitted in averaging.

Mixing.—All concrete was mixed in a small tilting drum mixer for 2 min. Enough concrete to mould three 6 by 12-in. cylinders constituted a batch. All materials were placed in the drum before starting the mixer. The mortar was hand-mixed.

Slump and Flow Tests.—Slump and flow tests were made on each batch, except for very wet ones, before the cylinders were moulded. The same students made the slump and flow tests on all concrete mixed during any one day. The drop of the flow table was ½ in. instead of the ⅛ in., as stated by Messrs. Slater and Walker.

Moulding.—One student did all the moulding during any one day's work. Part of the variation in strength is undoubtedly due to the variations in moulding by different individuals.

Curing.—All specimens remained in the moulds 24 hours. Immediately on removal from the forms, the cylinders were placed in the moist room and kept wet 27 days. Just before testing, the cylinders were removed from the moist room and capped with a fast-setting plaster of Paris. The age of all specimens at testing was 28 days, except in Problem 22, varying time of curing.

Testing.—All concrete specimens were broken in a hand-operated, 200 000-lb., hydraulic testing machine. The mortar specimens were broken in a 30 000-lb., motor-driven, knife-edge testing machine.

Results

Water-Cement Ratio.—Strength Relationship.—The results are given in Table 55 and Fig. 62. Each strength represents the average of three cylinders.

The results indicate that for the factors entering into these tests, a reduction must be made from the strengths indicated by the Abrams diagram.* The cement as received does not meet the specific gravity requirement. It is also deficient in the tension test. There has been some comment recently concerning the value of the tension test. The writer feels that at least part of the reason for lower strengths than those given by Abrams may be traced to the cement.

TABLE 55.

Problem 15, 1:2 Mortar by Weight, 2 by 4-Inch Cylinders.

Water-cement ratio.	Strength, in Pounds per Square Inch.				
	E.	*F.*	*G.*	*H.*	Average.
0.46	4 547	5 250	3 033	3 670	4 125
0.51	4 027	4 900	2 277	4 240	3 861
0.55	4 587	4 920	3 967	3 480	4 238
0.59	4 400	4 160	3 287	3 750	3 899
0.64	3 593	4 360	2 927	2 725	3 401
0.69	3 280	3 660	2 543	3 490	3 243
0.73	2 885	2 920	2 160	3 415	2 845

* *Bulletin 1,* Lewis Inst.

TABLE 55.—(*Continued*).

PROBLEM 23: MIX, FINE AGGREGATE AND CONSISTENCY VARIED.

Water-cement ratio.	Mix, cement to aggregate by volume.	Relative consistency.	Strength, in Pounds per Square Inch.					Slump, in Inches.					Percentage of Original Flow, Diameter.				
			E.	*F.*	*G.*	*H.*	Average.	*E.*	*F.*	*G.*	*H.*	Average.	*E.*	*F.*	*G.*	*H.*	Average.
0.56	1:3	1.0	4 250	3 330	3 120	3 787	3 622	¾	2½	6	1½	2¾	125	160	200	160	161
0.62	1:3	1.1	.....	2 680	.....	.....	2 680	...	7	...	...	7	...	205	...	...	205
0.64	1:4	1.0	3 200	3 180	3 080	2 890	3 057	6	4	5	6	5¼	180	165	200	170	179
0.68	1:3	1.2	3 130	.....	2 597	2 963	2 897	8	...	...	8	8	225	...	260	225	238
0.76	1:5	1.0	2 640	2 690	2 447	2 520	2 574	5½	2½	4	7	4¾	205	185	180	185	189
0.79	1:4	1.2	1 930	2 840	.....	2 073	2 280	8	9	...	8½	8½	235	250	...	245	243
0.84	1:5	1.1	.....	2 590	.....	.....	2 590	...	7	...	...	7	...	210	...	...	210
0.87	1:6	1.0	1 555	2 110	1 903	2 043	2 281	4	¾	3½	4½	3¼	195	160	195	205	189
0.93	1:5	1.2	1 830	.....	1 867	1 883	1 860	8	...	...	9	8½	250	...	240	245	245
0.97	1:4	1.46	.....	.....	1 727	.....	1 727	...	...	...	...	...	...	...	...	...	...
1.06	1:6	1.2	1 340	1 750	1 290	1 333	1 428	8	6	...	7	7	240	205	250	240	234
PLASTERING SAND AS FINE AGGREGATE.																	
0.64	1:4	1.0	2 720	2 880	2 447	2 510	2 639	5½	2	6¾	6	5	200	155	210	175	185
0.79	1:4	1.2	2 160	2 710	2 387	1 783	2 260	8¼	7½	...	9	6¼	240	220	240	250	238
0.87	1:6	1.0	1 825	2 160	2 080	2 123	2 047	1	¼	1	1¼	1	180	150	170	170	168
1.06	1:6	1.2	1 580	1 400	1 603	1 703	1 572	5½	6½	5½	2½	5	230	210	220	200	215

PROBLEM 20: MIXING WATER VARIABLE; MIX, 1:3.88.

Water-cement ratio.	Strength, in Pounds per Square Inch.					Slump, in Inches.				Percentage of Original Flow, Diameter.			
	E.	*F.*	*G.*	*H.*	Average.	*E.*	*F.*	*G.*	*H.*	*E.*	*F.*	*G.*	*H.*
0.58	2 480	3 590	3 217	1 928	2 804	0	...	0	0	140	...	130	115
0.67	2 980	3 660	1 570	2 953	2 791	0	...	5	¼	165	...	180	140
0.77	2 590	2 660	2 110	2 330	2 422	6	...	...	7¼	220	...	210	195
0.86	2 580	2 230	1 627	1 788	2 056	8	...	...	8½	260	...	...	230
0.95	1 980	2 220	1 260	1 348	1 702	7½	...	...	9¼	270	...	...	245
1.04	1 240	1 530	1 013	1 231	1 253	8	...	...	...	270	...	...	...
1.14	1 196	1 370	978	1 010	1 138	...	...	...	...	...	...	...	...
1.33	719	970	827	918	846	...	...	...	...	...	...	...	...

NOTE.—Except for Column *F*, blank spaces under slump and flow indicate that the student considered the batch too wet to be tested,

Apparently, the opinion is quite general that it is always possible to obtain certain pre-determined strengths. Although there is no question about the part played by water in determining strength, other factors, such as the quality of the cement and the aggregates, mixing, placing, and curing, must not be forgotten. These other factors may affect the strength as much as the mixing water. The writer believes that conditions vary widely on different work and that, as a result, the concrete strengths will vary widely. These variations can be eliminated to some extent as the number of engineers trained to supervise this work increases.

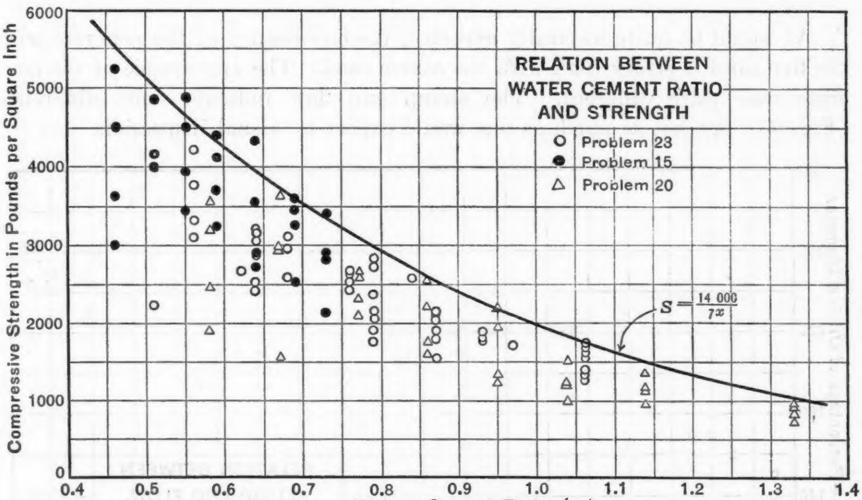

FIG. 62.

Apparently results of attempts in the field to use any particular method of proportioning are published only when the work has been successful. This is unfortunate. Even if the great majority of concrete work may be good, engineers should be prepared at any time to obtain variations. Check tests should be made wherever the nature of the work warrants.

Part of Problem 23 was to check the statement of Abrams that:

"The grading of the aggregate may vary over a wide range without producing any effect on concrete strength, so long as the cement and water remain unchanged. The consistency of the concrete will be changed, but this will not effect the concrete strength if all mixes are plastic".*

The average values from Problem 23 are as given in Table 56. The statement quoted is substantially borne out. The fine sand, however, gives a little less strength, using the same quantity of cement and water. The fineness modulus of the fine sand is less than that for the coarse sand. The percentage of fine aggregate was decreased when the fine sand was used, in order that the fineness modulus of the combined aggregate would be the same for each mix.

* *Bulletin 1*, Lewis Inst., p. 20.

TABLE 56.

Water-cement ratio.	Strength, in Pounds per Square Inch, Fine sand.	Strength, in Pounds per Square Inch, Coarse sand.	Slump, in Inches. Fine sand.	Slump, in Inches. Coarse sand.	Flow, Percentage of Original Diameter. Fine sand.	Flow, Percentage of Original Diameter. Coarse sand.
0.64	2 639	3 057	5	5¼	185	179
0.79	2 260	2 280	6¼	8½	238	243
0.87	2 047	2 281	1	3¾	168	189
1.06	1 572	1 428	5	7	215	234

As would be quite naturally expected, the consistency of the concrete with the fine sand is stiffer than with the coarse sand. The appearance of the concrete was quite different. The slump and flow indicate some difference (Fig. 63), but not as much as one would expect by visual inspection.

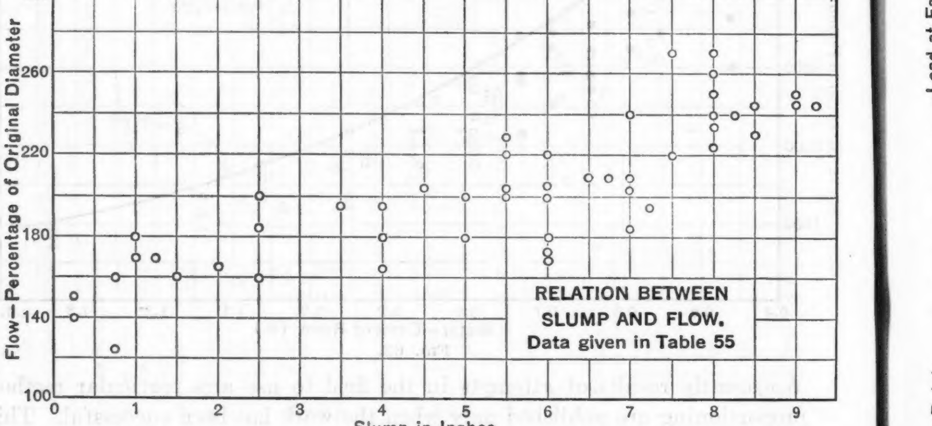

FIG. 63.

Method of Curing.—Strength Relationship.—Curing conditions were varied to show principally the effect of the absence of curing water. The results are shown in Table 57.

TABLE 57.—METHOD OF CURING. STRENGTH RELATIONSHIP.

Method of curing.	Strength, in Pounds per Square Inch. *E.*	*G.*	*H.*
One day in air; 27 days in moist room	3 135	3 840*	2 980
One day in air; 27 days under water	3 217	1 872†	3 020
Twenty-eight days in laboratory air. No moisture	2 532	1 635‡	1 960

* Thirty-four days in moist room.

† Thirty-four days under water.

‡ Thirty-five days in laboratory air.

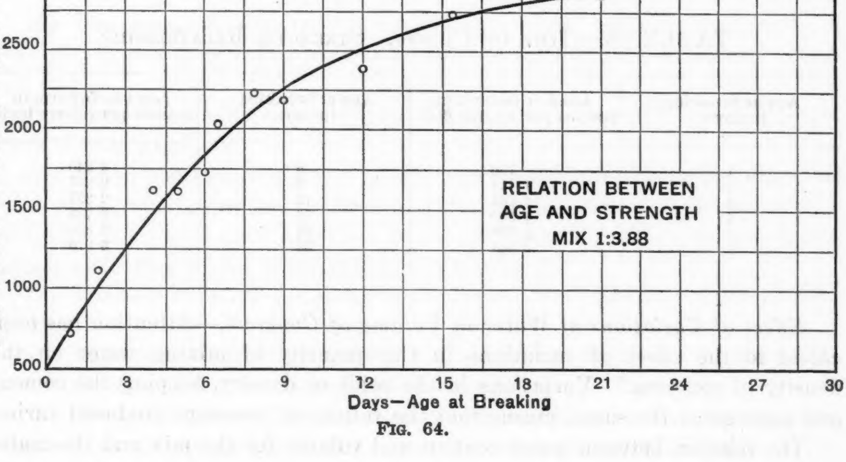
RELATION BETWEEN
AGE AND STRENGTH
MIX 1:3.88
Load at Failure in Pounds per Square Inch
500
1000
1500
2000
2500
3000
0
3
6
9
12
15
18
21
24
27
30
Days—Age at Breaking

FIG. 64.

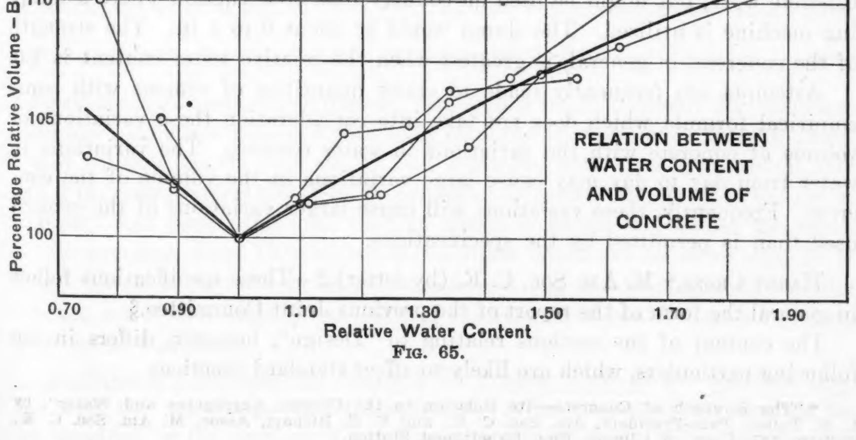
RELATION BETWEEN
WATER CONTENT
AND VOLUME OF
CONCRETE
Percentage Relative Volume – Batch
100
105
110
0.70
0.90
1.10
1.30
1.50
1.70
1.90
Relative Water Content

FIG. 65.

There does not seem to be much difference between the immersion and sprinkling methods. Lack of water during the curing period causes a reduction in strength of approximately one-fourth to one-third of the strength the concrete might have, if kept wet.

Age.—Strength Relationship.—The relationship between age and strength is shown in Table 58 and Fig. 64. Only one set of tests with this variable was made.

TABLE 58.—TIME OF CURING. STRENGTH RELATIONSHIP.

Age at breaking, in days.	Load at failure, in pounds per square inch.	Age at breaking, in days.	Load at failure, in pounds per square inch.
1	706	8	2 240
2	1 113	9	2 207
4	1 605	12	2 397
5	1 597	16	2 733
6	1 738	21	2 870
7	2 045	30	3 010

Effect of Variations of Water in Volume of Concrete.—Attention has been called to the effect of variations in the quantity of mixing water on the density of concrete.* Variations in the voids or density, keeping the cement and aggregates the same, means that the volume of concrete produced varies.

The relation between water content and volume for the mix and the materials used in Problem 20 are shown in Fig. 65. The water content that produced the minimum volume of concrete is designated as a relative water content of 1.0. Other relative water contents are based on the water content that produced minimum volume. Relative volumes are based on the minimum volume produced.

Batches with a relative water content of 1.0 are too dry for reinforced concrete work, but might be used on country road construction where a tamping machine is utilized. The slump would be about 0 to ½ in. The strength of the concrete, in general, is greatest when the relative water content is 1.0.

Attempts are frequently made to check quantities of cement with some empirical formula which does not take into consideration these variations of volume of concrete with the variations in water content. The variations in water from day to day may cause large variations in the volume of the concrete. Frequently, these variations will cause larger variations of the cement used than is permitted by the specifications.

HARDY CROSS,† M. AM. SOC. C. E. (by letter).‡—These specifications follow in general the form of the report of the previous Joint Committee.§

The content of the sections relating to "Design", however, differs in the following particulars, which are likely to affect standard practice:

* "The Strength of Concrete—Its Relation to the Cement Aggregates and Water", by A. N. Talbot, Past-President, Am. Soc. C E., and F. E. Richart, Assoc. M. Am. Soc. C. E., *Bulletin 137*, Univ. of Illinois, Eng. Experiment Station.

† Prof. of Structural Eng., Univ. of Illinois, Urbana, Ill.

‡ Received by the Secretary, June 19, 1925.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1101.

(*a*) Moment requirements in continuous beams and slabs:

(1) Under certain conditions the center moment is to be taken as $\frac{w\,l^2}{16}$ instead of $\frac{w\,l^2}{12}$ (Section 110).

(2) The center moment for end spans of continuous slabs is $\frac{w\,l^2}{10}$ (Section 110). The design difficulties are obvious.

(*b*) The required spacing of bars, which often controls the width of T-beams, is less than that specified by the previous Joint Committee. (Section 63.)

(*c*) The overhanging slab width for T-beams is increased from six to eight times the thickness.

(*d*) Under certain conditions it is permissible to use twice the allowable shear stress now permitted. (Section 128.) Thus, for 2 000 lb. concrete, a maximum computed shear of 240 lb. per sq. in. may be used, on condition that special anchorage of the reinforcement is provided.

(*e*) The requirements for shear reinforcement are elaborate and very complicated.

(*f*) The requirements for bond are very complicated. Under certain conditions there is no limit on the calculated bond stress.

(*g*) The total moment in flat slabs is 0.09 instead of $\frac{1}{8}$, but the computed compressive stresses are apparently increased. (See Formulas (40) and (41).)* The whole of Section *F* has a great air of mystery and accomplishes its ends with curious indirection.

(*h*) The whole existing practice in regard to columns is thrown overboard and a complicated and questionable set of rules and formulas is substituted, the most notable points of which are:

1.—Columns having a length of more than forty times the least radius of gyration are to be designed by a straight line reduction formula which gives a zero resistance for $\frac{l}{r} = 160$. A minimum diameter of 12 in. is specified for principal columns, as at present, although 6 in. is allowable for secondary columns. It is not required that long columns be spiralled. Thus, a 12-in., round column, 30 ft. long, of 1 : 2 : 4 concrete, with four $\frac{1}{2}$-in. round rods and $\frac{1}{4}$-in. round lateral ties, 8 in. on centers, will satisfy the specification for an ordinary roof panel, the total load on which is 20 000 lb. Under the report of the previous Joint Committee, this column could not be more than 15 ft. long. There is no need to be concerned over any bending moments in this column, for if they exist the amount of reinforcement can be doubled and the basic stress increased.

2.—Spiralled columns are designed for a basic stress in the concrete varying with the percentage of reinforcement. A maximum of 6% against the 4% specified by the previous Joint Committee, is allowed. Thus, the previous Joint Committee allowed 700 lb. per sq. in. in the concrete for 2 000 lb. con-

* *Proceedings*, Am. Soc. C. E., October, 1924, Papers and Discussions, pp. 1202-1203.

crete for all percentages of steel, whereas the present Joint Committee allows stresses varying from 580 lb. for 1% to 980 lb. for 6 per cent.

3.—The protection of spiralled columns is exclusively specified in Section (164). In Section (165), A_c' is defined as "the net area of concrete in the column", whereas in Section (162), A' is the "area of the concrete core enclosed within the spiral". The inclusion of the protective covering for tied columns is clearly implied. Spirals are thus penalized for small columns.

4.—Where concrete is used in connection with structural steel, a liberal allowance (500 lb. per sq. in. for 2 000 lb. concrete) is made for both enclosed and encasing concrete if the latter is spiralled. The slenderness ratio for such steel columns is not limited, except as the formula gives zero resistance for $\frac{l}{r} = 240$.

5.—It is definitely required that "bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint and shall be provided for in the design". Liberal increases in stress, however, are permitted in Section (167).

(*i*) An elaborate chapter on footings has been added, which presents common practice considerably mangled—and not improved in attractiveness—by the hoofmarks of several hobby-horses.

(*j*) A chapter—largely innocuous—is added regarding reinforced concrete retaining walls.

(*k*) The allowable compression in beams has been increased 25 per cent.

(*l*) The section on the design of slabs supported on four sides is omitted.

The writer thinks that many sections in these specifications are not warranted by published data. The shear sections are based, perhaps, on the tests made by the U. S. Shipping Board.* Data on these tests are not published and engineers should not be asked to accept conclusions based on reasons not stated and deduced from unpublished data. It may be true—in fact, it is probable—that higher shear values may be used if slip of the bars can be prevented. Doubtless many—very many—shear failures in the past have been due to bond failure; but tests have been misinterpreted and misunderstood before this. Probably higher shear values may be used; possibly there is no limit on calculated bond if the bars are anchored; perhaps even 12-in. columns can safely be 48 ft. long—but let us examine the data.

These specifications break radically with practice and precedent in America. The Joint Committee might have been less specific in many places, for the elaborate detail and dictatorial phrasing put the designer off his guard; or else the specifications should have been—in view of the important changes needed in practice—much more detailed and specific, more so, perhaps, than lack of human hindsight as to the future makes possible.

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* In a paper entitled "High Points of the New Joint Committee Report on Concrete", *Engineering and Contracting*, May 27, 1925, N. M. Stineman, Assoc. M. Am. Soc. C. E., says they are based on "investigations and tests conducted by the U. S. Bureau of Standards". The writer did not previously know of the tests by the Bureau of Standards.

The writer's objection is not that inconsistencies exist; they are probably inevitable in any specification, but he believes that the Joint Committee has attempted what is most unwise as regards the future and apparently impossible as regards the present. It has attempted to make of concrete design a poor slave of rules, formulas, and dogmas, whereas what is needed is maximum freedom for development consistent with safety.

"* * * concrete and reinforced concrete involve the exercise of good judgment to a greater degree than do any other building materials. Rules cannot produce or supersede judgment". * * * "Only persons [having an adequate knowledge of the principles of structural design] should be called upon to design reinforced concrete structures".*

The first Joint Committee apparently saw the designer's work in a broader way than this Committee and, in consequence, presented a report less mandatory, less detailed, and, at the same time, more conservative and more comprehensive. The following examples further indicate this view.

104 and 113.—*Flexure Formulas.*—These formulas are no longer needed. They might be included in an Appendix. Engineers might as well give formulas for bridge stresses in a bridge specification.

It may be noted in passing that the formulas for double reinforced beams are incorrect, in that the concrete replaced by the compressive steel is not deducted.

106.—*Span Length.*—Why specify that the span length "shall not exceed", "shall be the clear distance", "shall be measured"? The first Joint Committee says "may be". This is surely a better wording. The wording, "shall be taken as", occurring later in connection with the unsupported length of concrete columns is harmless, as the detailed rules there given state neither more nor less than that the unsupported length of a column "shall be taken as" its unsupported length, to which no one can object.

122.—*Variation of Shear in Beams with Uniform Load.*—This section seems at first glance to be an improvement over present practice, at least in theory, but the theory of shear reinforcement is almost wholly empirical and the reduced stress and bond in the longitudinal reinforcement probably has a greater effect than the increased shear. Such a view certainly seems more nearly in accordance with the views of the Committee as stated in Section 128.

The section is awkwardly worded. It probably is not intended to apply to cantilever beams, although as it is worded, they are included.

123.—*Width of Beams in Shear Computations.*—This section is obviously incorrect if the minimum width occurs on the compression side of the neutral axis, which is not at all an uncommon case. This illustrates the necessity for a statement of fundamentals rather than an attempt to cover all possible cases and exceptions.

125.—*Formula (30) for Inclined Bars.*—This is a labored effort to rationalize certain strictly empirical data. The same is true of Formulas (31) and (32), where obviously there should be no discontinuity at 45 degrees. This effort to make exact an approximate control is futile.

* Final Report of the Special Committee on Concrete and Reinforced Concrete, *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1108.

118.—*Flange Width in Shear.*—Surely the flange is effective in computing the shearing resistance. For further indication as to the meaning of the flange, see Section 115.

134.—*Critical Section for Pile Footings.*—This statement is certainly very complex; also, it presupposes an understanding of more things than any one can possibly know.

182.—*Transfer of Stress (Metal Distributing Bases).*—The whole section is complex: "When metal distributing bases are used, they shall have sufficient area and thickness to transmit safety the load from the longitudinal reinforcement in compression and bending". The writer has never used metal distributing bases, but finds difficulty in picturing their failure in ordinary construction.

I.—Reinforced Concrete Retaining Walls.—In Items (*a*) and (*b*), the first two details of design specified certainly represent remarkable clauses; to be consistent the Joint Committee should also specify what area and perimeter shall be used for circles.

In Item (*c*) no good reason is given why slabs supported by counterforts or buttresses should be designed for a total moment greater than $\frac{1}{8}$ *Wl*.

Referring to Items (*e*) and (*f*), it is certain that the laws for prismatic beams do not apply to wedge-shaped beams.

The points discussed in Items (*d*) and (*j*) necessarily depend so much on the judgment of the designer that they might well be omitted.

107 to 111.—*Moment Coefficients.*—

(*A*) Why not at least permit the computation of these moments in regular as well as in irregular cases?

(*B*) The wording is peculiar. There is a specification for slightly restrained beams of equal span "which carry uniformly distributed loads"; beams built into brick or masonry walls (irrespective of kind of loading); freely supported beams of equal span, "assumed to carry uniformly distributed loads"; restrained beams "assumed to carry uniformly distributed loads"; and continuous beams of unequal span or with non-uniform loading.

(*C*) In Section 107, Paragraphs (*c*) and (*d*) call for different moments in slabs of exterior and interior panels of a slab and girder design. This is not practical. The first Joint Committee calls for $\frac{w\,l^2}{12}$ in both cases.

(*D*) It is difficult to understand what is meant by Section 110(*a*). Successive beams of a floor usually frame into columns and girders: The former come under Section 110 (*a*) with a center moment, $\frac{w\,l^2}{16}$, and the latter under Section 107 (*c*) with a center moment, $\frac{w\,l^2}{12}$. In practical design they will be similar. All girders frame into columns and, perhaps, it is intended that these shall be "assumed to carry" uniform load. If designed under Section 111, they might carry in a certain case: $+\frac{1}{10}\,W\,l$, or $-\frac{1}{8}\,W\,l$. Just how shall they be designed?

(*E*) The coefficients given are presumably for ordinary ratios of live load to dead load, say, 3 : 1 to 1 : 3. Extraordinary ratios as, for example, all dead loads, should also be included in Section 111. One wonders whether this clause will affect the common practice—which is wrong—of taking two-thirds of the maximum positive moment on a simple beam for both maximum positive and negative moments, or whether it is intended that it should do so.

111.—*Negative Moments in Short Spans Adjacent to Long Spans.*—Compare the first Joint Committee wording "more exact calculations shall be made" with that used in this Section, "shall be designed for the actual moments under the conditions of loading and restraint". The latter is quite an order for the average engineer.

In connection with the second paragraph, note that in equal spans simply supported it is theoretically possible and actually probable that the beams will be loaded so that there will be no point of inflection in a span (negative moment existing entirely across the span), provided the live load equals or exceeds the dead load. The existence of negative moment across a span is not necessarily a dangerous condition, although its limits of safety have not been defined in any specification.

For some reason the Joint Committee specifies that these negative moments "near the center" need be "provided for" only in short spans which are adjacents to long spans, shortness and length not being defined. That they are much more frequent due to high intensities of live load on short spans is not mentioned. Here, as elsewhere, any form of construction which is not perfectly regular is penalized, and the method of calculation wags the design.

135 to 141.—*Bond and Anchorage.*—The sections on bond and anchorage are the most important perhaps in the whole specification, for on them seems to be founded the whole theory of increased shear values, and these values, together with the closer spacing allowed for reinforcement, effect pronounced economies in concrete beams. The writer does not object to the closer spacing of reinforcing bars on any ground except that no evidence has been presented that such an innovation is conservative. The present rules are certainly based on poor logic and seem to have inadequate support from tests; but what logic and tests support the new rules? As regards the increase in shear values, the writer knows of no good reason why there need be any limit on shear values, provided the diagonal tension is cared for by inclined steel adequately anchored, the diagonal compression is not excessive, and the longitudinal reinforcement does not slip. All this, of course, depends on what constitutes adequacy of anchorage; the solid concrete steel truss in a beam is a safe structure if the connections (bonds) hold; but what is the basis for all the complexity as to shear?

Similarly, every one realizes that a bar which over-runs its point of theoretical zero stress will have bond on the over-run, but it is radical to state, as does the Joint Committee, that it makes no difference what the calculated bond is at the point of theoretical zero stress, provided the total bond surface

satisfies Formula (35). Study of these Sections 135 to 141, inclusive, indicates clearly that the rules are not even semi-rational. Committee E-1 of the American Concrete Institute, which is codifying—and modifying—the Joint Committee report states that the allowable calculated bond shall be increased only 150 per cent. Where did the Joint Committee get the rule that the calculated intensity of bond makes no difference if the total bond is provided for? Is this based on the U. S. Shipping Board tests made for a special purpose during the pressure of war construction? If so, where did Committee E-1 get its limitation?

135.—*Bond at Point of Inflection.*—"The critical section for bond shall be assumed to be at the point of inflection". This is meaningless, unless the condition of loading is specified. Perhaps what is intended is that the critical section for bond shall be taken at that point of contraflexure which exists for the condition of loading giving maximum stress in the positive and negative reinforcement, respectively. There are then two points of inflection the location of which vary with the stiffness of columns and the ratio of live to dead load.

Evidently something quite definite is intended, as the point of inflection is impliedly "locatable" within a distance, $\frac{d}{3}$. Perhaps the quarter-point for negative and the eighth-point for positive moment may be accepted as a reasonable compromise. Whatever is intended, one is left to wonder what peculiar phenomenon makes the point of inflection significant in bond computations. At best, the theory of bond is a very rough approximation, and little better than rule-of-thumb, the fundamental allowable stress being uncertain. Even those so-called beam tests of bond, where the central part of the beam is cut away, differ from the pull-out and push-out tests only in changing from a reasonably determinable condition of stress in the surrounding concrete to an indeterminable condition which is quite different from that in a beam.

176.—*Critical Section for Bending.*—Formula (48) is apparently based entirely on *Bulletin 67* issued by the Engineering Experiment Station of the University of Illinois. When the Joint Committee specifies that "the bending moment * * * shall be computed from the load on the trapezoid" described, and from this loading attempts to deduce Formula (48), which is the same as Formula (27) of the *Bulletin* mentioned, it goes further than its authority, as a careful study of page 19 of the *Bulletin* will at once show.

179.—*Irregular Footings.*—This section is both ambiguous and arbitrary. What "rectangles and trapezoids" (triangles are apparently barred) are tributary to the sides of the column? May one side of the trapezoid extend across the slab? The method used in the case of regular footings would suggest the use of a trapezoid with a side of the column as one base. This, however, is incorrect in that it violates the laws of statics and is certainly not supported by any tests on irregular footings.

Would it not be wiser, in the interests of uniformity, if for no other reason, to design all footings for the static moments? These specifications go much

further in requiring the use of Formula (48) than the data on which they are based.*

The formula is there merely suggested with reservations. In the *Bulletin* mentioned about 70% of the moment given by statics, on a section through the face of the column, is used to compute the stress in $\frac{46}{60}$ of the bars. Obviously, the result is practically the same as considering that all bars take the full static moment on this section. In the reinforced column footings which failed in tension, the ratio of the stress in the steel at failure, as computed by statics, to the yield point of the steel is about 115 per cent. If the section of maximum moment be used, the ratio is about 130 per cent. Moreover, the modulus of rupture of unreinforced concrete footings, as there determined, checks quite closely with the modulus of rupture of the control beams. Accepting the value of "perhaps two-thirds if some allowance be made for the greaterage of the control beams"† for the ratio of the modulus of rupture of the footings as computed from Formula (48) to that of the control beams, and multiplying by the ratio of the moment computed by statics on a section across the column face to that computed by Formula (48), the following is obtained, $\frac{2}{3} \times 147\% = 98\%$, for the ratio of the average modulus of rupture of the footings to that of the control beams, "if some allowance be made for the greaterage of the control beams".

Surely there can be no objection on the grounds of these tests to compute tension stresses in footings by statics. To do so adds simplicity and uniformity; indeed it makes Sections 176, 177, 178, and 179 unnecessary.

In Section 178 it is specified that the extreme fiber stress in the concrete "shall be based on the exact shape of the section for a width not greater than that assumed effective for reinforcement". What is meant by the exact shape of the section? Is it necessary to calculate the stress in an irregular section? This is inconvenient and apart from any theory which the Joint Committee may hold as to the action of sloped or stepped footings, it is far from exact in the case of any beam with non-parallel faces, as William Cain, M. Am. Soc. C. E., has indicated.‡ Also, it seems certain that the stress at the top of the footing is not tensile at points beyond the neutral axis as here computed. Indeed, all the recommendations as to sloped and stepped footings seem ill-advised.

It is believed that the specifications would be improved by the omission of the whole section on "Footings," with the exception of the discussion of column bases which more properly belongs with the sections on "Columns".

160 to 171.—*Reinforced Concrete Columns.*—The theory of concrete column design based on the effects of shrinkage and time yield will be recognized by the profession as an interesting hypothesis, as old as Considère, not wholly substantiated by tests nor disproved by them, and, because it deals with highly

* *Bulletin 67*, Eng. Experiment Station, Univ. of Illinois.

† *Loc. cit.*, p. 72.

‡ "Stresses in Wedge-Shaped Reinforced Concrete Beams", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 745.

complex phenomena occurring near the yield point of the steel, not readily amenable to a complete rational analysis. It correctly recognizes that phenomena at working stresses are of no interest whatever to the engineer except in so far as they indicate the factor of safety against failure and that when, as is alleged in the case of columns, the conditions at failure differ radically from those at working stresses, the latter have no significance whatever. It is certainly not true, however, that the theory has the uniform support of the profession.

Assuming that the chief function of the concrete in preventing failure in a concreted steel column is to relieve the steel when the latter has reached its yield point, it is difficult to reconcile this theory with the treatment of structural steel columns surrounded by concrete, for the failure of the steel may be by buckling with consequent rupture of the concrete shell and enclosing spiral and not by gradual yielding, as is assumed in the case of the spiralled column with longitudinal reinforcement. The whole question is intricate, and it is not desirable to elaborate its complexities. The dogmatism of the specifications, however, is in marked contrast to the caution voiced by the first Joint Committee in regard to such columns. In this regard these specifications may be said to be lawless as regards American practice.

Practically nothing is known in regard to long concrete columns. Columns are weaker than short struts because they are likely to buckle. Buckling may result from the inherent elastic instability of compression members or from dissymmetry resulting from eccentric loading or lack of homogeneity. The elastic instability is amenable to law, and a series of Euler's columns may show a fairly definite relation between strength and slenderness; on the other hand, the dissymmetry is largely lawless. If, then, the elastic buckling is the controlling factor, a rational formula, such as that of Euler, applies fairly well; if the accidental eccentricities play a large part, as is the case in steel columns, some semi-rational formula, such as the Rankine-Gordon, applies, but less exactly; if the accidental and lawless elements control, as in concrete columns, then the phenomena are lawless. In such a case, safe limits may be established by a long series of tests, but no law will be found where none exists. Moreover, it is a long jump from the small laboratory column to the large column on construction work. Some attempts to determine the effect of slenderness ratio have only shown that, in general, the larger the column the smaller is the danger from lack of homogeneity.

This criticism of the Joint Committee's effort to rationalize the buckling of concrete columns applies also to the semblance of scientific accuracy produced by substituting the radius of gyration for the familiar diameter in Section 160.

In relation to bending in columns the Joint Committee follows more or less closely the wording of the first Committee's report, except that it has specified that all bending moments "shall be determined on the basis of loading conditions and end restraint", whereas its predecessor impliedly included only such obvious cases as occur with "unequal spans or lateral forces".

In addition, the specification permits an increase of 20% in the concrete stresses allowed for the combination of compression and bending. In

the steel, it is specified that "the tension due to bending shall not exceed 16 000 lb. per sq. in." This statement seems very unreasonable if it means what it clearly says, that the compression on the column shall be neglected in computing the steel stress.

142 to 159.—*Flat Slabs.*—These slabs, as usual, are cloaked in a veil of mystery, which the foot-note to Section 142 only partly lifts. Many rules covering the design of flat slabs are necessarily empirical, but engineers have progressed in their methods of thinking of the moments in these structures since the paper* by John R. Nichols, M. Am. Soc. C. E., in 1914, and since the test by A. N. Talbot, Past-President, Am. Soc. C. E., on the Western Newspaper Union Building in 1917.† The total moments in flat slabs are given by statics, and it would be well for the Joint Committee so to state. As regards clarification, therefore, the treatment surely is a step backward from that of the first Joint Committee in its Final Report. Tests and experience, however, indicate that the factor of safety for these structures may be reduced.‡ If this is really the correct basis for allowing increased fiber stresses (or decreased moments) in flat slabs, then the subsequent increase in calculated stress in the concrete seems groundless. The writer believes that it is not advisable and will give an unbalanced design.

The writer believes also that the exact specification of the moment coefficients to be applied to different parts of the slab is not justified by the action of the slabs under load. In so far as strain-gauge measurements made on flat slabs in the field can be quantitatively interpreted, they indicate that the exact distribution of the steel is not especially important in determining the load which will cause failure (not collapse).§ Tests made at loads below those producing failure do not seem significant in determining the factor of safety as the stress is not theoretically even approximately proportional to the load until the concrete is thoroughly cracked and the measured strain is not proportional to the maximum stress because the bond of the concrete between cracks materially affects the ratio of average strain to maximum.

The phenomenon of redistribution of stress in the reinforcement of slabs appears in the case of footings,‖ of slabs supported on four sides,¶ and in other cases, and its existence in such cases seems so probable as scarcely to need the support of test data. A clear appreciation of its existence indicates that more tolerance as to the arrangement of steel should be permitted.

It is difficult to understand why any distinction should be made between flat slabs and footings in allowing for the thickness of the slab in computing

* "Statical Limitations Upon the Steel Requirement in Reinforced Concrete Flat Slab-Floors", *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 1670.

† *Bulletin 106*, Eng. Experiment Station, Univ. of Illinois.

‡ For an illuminating statement of this view, see "Principles of Reinforced Concrete Construction," by Turneaure and Maurer, Edition of 1919, p. 288.

§ See, for example, "Test of a Flat Slab Floor of the Western Newspaper Union Building", by A. N. Talbot and H. P. Gonnerman, Univ. of Illinois Eng. Experiment Station *Bulletin 106*.

‖ "Reinforced Concrete Wall Footings and Column Footings", A. N. Talbot, Past-President, Am. Soc. C. E., Univ. of Illinois Eng. Experiment Station, *Bulletin 67*, p. 106.

¶ "Moments and Stresses in Slabs", H. M. Westergaard, Assoc. M. Am. Soc. C. E., and W. A. Slater, M. Am. Soc. C. E., *Proceedings*, Am. Concrete Inst., Vol. 17, Section IV, by Dr. Westergaard.

the critical section for shear. It is believed that Sections 131 and 133 can be made more consistent.

Conclusion.—These examples may serve to indicate the shifting viewpoint of the Joint Committee. The writer, however, does not wish to be misunderstood. He recognizes that many of the innovations of this report are extremely valuable, if properly presented. In the past, the reports of the Joint Committee have carried great authority; it is important that, if the Society is to co-operate in this work, they should continue to do so. The writer is not discussing codes based on this report, nor interpretations prepared by some association or other. Here is a report which is supposed of itself to present the best thought of American concrete engineers, yet only a few men know or can find out what are the bases for many of its recommendations. It is safe to say that many members of the Committee do not know.

The writer had hoped for progress in clarifying several subjects. In a list of more than fifty detailed criticisms of the Progress Report of 1921, which he wrote and which was presented to the Joint Committee, the writer especially pleaded for:

(*a*) Less formularization, especially in the formulas for shear and bond (Formulas (29) and (34)), which are so obviously applicable only to prismatic beams.

(*b*) A broader rule for the computation of bending moments, so that in all cases it would at least be permissible to use the elastic theory as a guide in design.

(*c*) A rational treatment of flat slabs and footings.

Many of the detailed criticisms were met by revisions in the Final Report, but the broader objections remain. These criticisms are not met by saying that the function of the Joint Committee was to write a specification—which is true, and is also unfortunate—and not a textbook. The treatment of these subjects could be made clear without being didactic.

This Committee has worked at an enormous expenditure of time and money and it is not creditable to the profession that so little discussion of its work has appeared in print. For this the Committee is partly to blame, as regards some of the design clauses, as it is necessary to find the bases for its conclusions by private conversations with its members or from internal evidence, and the bases thus found are often divergent. The present tendency is to revise all building loads downward and all building stresses upward and with this tendency the writer is in sympathy if designing engineers and draftsmen can be better educated as to the logical and experimental basis of the revisions. Concrete design is, however, now at the stage where, to paraphrase the words used by an eminent concrete engineer in connection with the Final Report of the Special Committee on Stresses in Structural Steel:* "Until such education can be attained, it is unsafe to adopt higher unit stresses and they have no application in ordinary building construction practice."

* *Proceedings*, Am. Soc. C. E., May, 1925, Papers and Discussions, p. 930.

Only full and free discussion can relieve this condition of inadequate understanding of the fundamentals of the report. For it will bear repetition that: "Concrete and reinforced concrete involve the exercise of good judgment to a greater degree than do any other building materials. * * * Only persons [having an adequate knowledge of the principles of structural design] should be called upon to design reinforced concrete structures."

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

CHARLES HINCKLEY BAKER, M. Am. Soc. C. E.*

DIED APRIL 18, 1924.

Charles Hinckley Baker, the eldest of the five children of William Taylor and Eliza (Dunster) Baker, was born on November 30, 1864, at Chicago, Ill. His parents were natives of the States of New York and New Hampshire, respectively, and were both of English descent. Henry Dunster, the founder of the Dunster family in America, was the first President of Harvard University.

Charles Hinckley Baker was graduated with honors from Cornell University in 1886 with the degree of B. C. E. After one year of work on railroad construction in Dakota for the Chicago Northwestern Railway Company, he accepted a position in Seattle, Wash., in the same kind of work, with the Seattle, Lake Shore, and Eastern Railroad Company.

After three years Mr. Baker resigned from this position and opened a private engineering office in Seattle, and soon developed a contracting business for the construction of railroads and water-works. In 1891, he built, by contract, the Third Street and Suburban Electric Railway, in Seattle, and an electric light and power plant in connection therewith. Later, he was appointed Receiver of the Merchants National Bank of Seattle.

In 1898, he planned and constructed at Snoqualmie Falls the first hydro-electric power plant in the Northwest and organized the Snoqualmie Falls Power Company (later, the Seattle Tacoma Power Company), of which he was President, General Manager, and Chief Engineer until the end of 1904. This plant was, and still is, extremely novel for the reason that the power-house consists of a cavern hewn in the rock about 300 ft. below the bed of the river. The Niagara Plant had just been constructed with the turbines placed in the shaft underground, and Mr. Baker carried his work even further than that at Niagara by placing the whole plant underground.

In 1904, he moved to New York, N. Y., and was one of the organizers of the Muscle Shoals Hydro-Electric Power Company and the Alabama Interstate Power Company and, as Engineer, made the first surveys and plans for the Muscle Shoals Project. He also organized and served as Vice-President of the American Cyanamid Company. He was largely interested in reclaiming about 100 000 acres of land in the Everglades District in Florida, on which tract the City of Moorehaven is now located.

Mr. Baker's principal professional interest was in the development of new, original, and unusual projects. The fact that these enterprises were carried

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* Memoir compiled by C. B. Wing, M. Am. Soc. C. E., with the assistance of H. L. Gray, M. Am. Soc. C. E., William W. Best, Esq., of Seattle, Wash., and H. R. Hoffeld, Esq., of Buffalo, N. Y.

through to a successful conclusion is due in large measure to his clearness of vision and careful organization.

Four children survive him, namely, William T. Baker, Leslie D. F. Baker, Mrs. Dorothy Baker Calhoun, and Ruth Baker.

Mr. Baker was elected a Member of the American Society of Civil Engineers on June 4, 1902.

JOHN HOFFMAN DUNLAP, M. Am. Soc. C. E.*

Died July 29, 1924.

John Hoffman Dunlap, son of George Harlan and Mary Catherine (Folger) Dunlap, was born in Harrisville, N. H., on September 9, 1882. He was educated in the public schools of Concord, N. H., and in Dartmouth College, from which he received the degree of Bachelor of Arts in 1905, and the degree of Civil Engineer from the Thayer School of Civil Engineering in 1908. Between 1905 and 1908, besides doing the work for his engineering degree, he served with the United States Reclamation Service, in Nevada, and for a time with the Pennsylvania Lines West, in Columbus, Ohio.

During the summer of 1908, Mr. Dunlap was Field Instructor in the Thayer School, and in the fall of that year he was engaged as Instructor of Descriptive Geometry and Drawing by the College of Applied Science of the State University of Iowa, with which institution he remained until his appointment as Secretary of the American Society of Civil Engineers, in 1922. He was made Instructor in Civil Engineering in 1909, Assistant Professor of Hydraulics and Sanitary Engineering in the Department of Civil Engineering in 1911, Associate Professor in 1917, and Professor in 1920.

During his stay in Iowa Mr. Dunlap was engaged in engineering work, as time permitted, was for seven years Secretary-Treasurer of the Iowa Engineering Society, and for one year its President, and he wrote and published more than twenty-five papers on various engineering matters, principally in the field of sanitary engineering practice.

He was also active in the formation of the Iowa Section of the American Society of Civil Engineers and was its President for one year.

Mr. Dunlap was a member of the Society for the Promotion of Engineering Education, the New England Water Works Association, the American Water Works Association, the American Public Health Association, the Proportional Representation League, Gamma Alpha, Phi Betta Kappa, Sigma Xi, and of the Sigma Nu Fraternity. During the later years of his life in Iowa City, he was President of the Social Service League of Iowa City, and a member of the Executive Committee of the Boy Scout Council.

On June 29, 1910, in Franklin, Vt., he was married to Fanny Maria Gates. She and three sons, George Spaulding, Richard Folger, and Clark Gates, are living.

* Memoir prepared by Anson Marston, R. W. Crum, and William G. Raymond, Members, Am. Soc. C. E.

Mr. Dunlap's death resulted from injuries sustained in a railroad wreck near Chicago, Ill., on June 30, 1924, as he was returning from the Annual Convention of the Society that had been held in Pasadena, Calif. He was buried in Franklin, Vt., on July 31, 1924.

Mr. Dunlap was elected an Associate Member of the American Society of Civil Engineers on April 17, 1917, and a Member on June 6, 1921. He was elected Secretary of the Society on June 19, 1922.

As almost all Mr. Dunlap's professional life was lived in the State of Iowa, it seems fitting to record first something of the position he won for himself there.

A session of the Iowa Section of the American Society of Civil Engineers was held in Iowa City on October 7, 1924, to honor his memory. At this session addresses were made by C. S. Nichols, M. Am. Soc. C. E., Professor at Iowa State College, and Past-President of the Iowa Section, Lloyd A. Canfield, Assoc. M. Am. Soc. C. E., a former student under Mr. Dunlap and Secretary of the Iowa Engineering Society, Anson Marston, M. Am. Soc. C. E., Dean of Engineering, Iowa State College, then Vice-President of the American Society of Civil Engineers, and William G. Raymond, M. Am. Soc. C. E., Dean of the College of Applied Science of the State University of Iowa. Excerpts from the addresses of Professor Nichols and Dean Raymond will indicate Mr. Dunlap's position in the State as an engineer, as a teacher, and as a citizen.

Professor Nichols spoke, in part, as follows:

"We are gathered here to-night, a company of loyal friends, with hearts and minds in perfect harmony, to pay honor to a departed comrade. How empty must sound any words we shall utter in our feeble attempts to express our admiration for and appreciation of John Dunlap. Much more appropriate would be a session of silent meditation together, mentally building him into our midst once more, and reviewing in retrospection the pleasant and profitable association we have had with him.

"John Dunlap was first of all a man, a Christian man whose every activity was in keeping with his noble principles and high ideals. He was an engineer, true, but only because through this profession he found the best opportunity to use his talents for service. To this end he became the best engineer possible. This was to him but natural, because it was fundamentally essential to insure a full measure of usefulness.

"He looked well to his tools, both as to manufacture and condition, and he became proficient in their uses. In his tool chest were some most valuable pieces of equipment. If we were to pick them out, one by one, we would probably choose first of all his Refinement of Thought. Who could ever forget his masterful address at Sioux City, in 1922, as President of the Iowa Engineering Society, on 'The Engineer's World', and this earnest appeal to his fellow engineers: 'Shall we not each of us dedicate ourselves, our lives, our training, our property, all that we have and are, in order that we may serve as good citizens of a noble country, and of a world growing steadily better, because it is an engineer's world in which each engineer is playing his part, with a firm and steadfast faith that the soul of civilization is not meat and raiment, but life and spirit.' Then and there he probably as nearly as ever bared his soul to his fellow men. This took courage, indomitable courage,

because it was a radical departure from the ordinary, and it was only because he was sound personally that he could say those things.

"And right there he showed the handiwork of another important tool, Definiteness of Purpose. Never, under any circumstances, could John Dunlap be swerved from the line of duty which he had set up for his life. Even as he pioneered, he kept the line straight, his compass in his hand. He had the courage to speak his convictions even though he stood alone.

"Did any of you ever fail to understand him when he spoke to you either in public address or in private conversation? I cannot conceive of it. He was master of those exceptional tools, Clearness of Thought and Clearness of Presentation. He had a wonderful ability to integrate ideas and impressions and to rationalize them into concrete expression, whether in speech, in illustration, or in construction. Through these characteristics he gained the finest confidence of those with whom he dealt.

"Diligence also was his constant aid, and explains in large part his wonderful capacity for accomplishment. Most things did not come easy for him; he worked as probably few of us ever will know how.

"Of him it can be said truthfully that he accepted Personal Responsibility for his every action. There was no passing of responsibility by John Dunlap. If he made mistakes, he acknowledged them, and made amends so far as it was physically possible. He was thoughtful and deliberate in his preparations, but when the time came to act there was brought into play an irresistible force for accomplishment. But his zeal for accomplishment and his earnestness in putting things through were ever tempered by an abundance of Consideration of Others. Even his opponents in controversy considered him a clean and honorable fighter for the principles he championed.

"Above all we like to recall his Cheerful Disposition and his saving Sense of Humor. Of his perplexities he kept us in complete ignorance, except as they were common to a considerable group and called for group discussion. Always bright and happy, his presence added joy to any gathering, large or small.

"These are the things which made him a man among men, and, coupled with native ability, made him a good engineer. We are but incidentally interested in the details of his professional work, as they contributed to his success only in the experience they afforded, and the opportunity they offered for contact with men and for the rendering of personal or community service. It is of interest, however, to follow through the high points of his work, and catch the point of view throughout.

"It was while acting as Field Instructor in the summer of 1908 at his Alma Mater that he saw the large opportunity for service in the intimate contacts which are possible between instructor and student, and decided to devote himself to educational work. He joined the Engineering Faculty of the State University of Iowa, where he rose in rank and influence until his call to the executive leadership of our National Society.

"During the summer of 1915, while engaged in a water-works survey for the Extension Division of the State University, he caught his first real perspective of conditions surrounding the administration and operation of municipal utilities, and saw the opportunity of and great need for those improvements which only the experienced engineer can direct properly. It was then that his personal opportunity and duty became plain. During that summer he visited about forty of our best Iowa cities, and while employed only to gather data for a report, he was diligent in his efforts to assist the local officials in remedying their difficulties. The full value of this voluntary advice and personal service will never be known, but there is much evidence that through his report and recommendations there resulted not only local improvements in many of the towns visited, but State legislation as well, which through the years to come will safeguard the interests of both municipal and private water-works companies.

"During the summer of 1916, he was Engineer in charge of the construction of the Sewage Treatment Plant for the State Hospital and Colony for Epileptics at Woodward, and there became impressed that an engineer's work was not complete until he had provided adequately for the future operation of the plant by the layman operator. He made a remarkably clear perspective sketch of this plant, showing every operating detail, accompanied by detailed instructions for its operation, and had them framed and hung in the plant. He also spent hours of his own time in providing for satisfactory supervision and maintenance.

"The summers of 1917 and 1918 were spent under the auspices of the State Board of Health inspecting sewage treatment plants in all parts of the State. In this capacity, he rendered the same valuable service to the municipalities that had characterized his work two years before in connection with the waterworks plants. Ever on the outlook for methods of making improvements, he formulated excellent recommendations for more complete supervision and control by the State Board of Health of the design, construction, and operation of municipal sanitary works.

"He was one of the most ardent supporters of the annual conferences of sewage treatment plant operators and contributed much to their success.

"In his consulting practice he invariably rendered his clients a service of high character. The esteem of a man by his own townspeople usually tells the true story. One instance in the case of John Dunlap will suffice. In 1915, a controversy arose between the Iowa City Water Company and the City as to water rates. The City retained John Dunlap as its engineer, and after investigation and consultation a satisfactory rate was agreed upon. For various reasons this rate continued throughout the World War period, although during this time the company is reported to have sustained a considerable loss. With continued high prices, therefore, some adjustments in the rate became necessary, and the Company chose John Dunlap to make a complete valuation of its properties and determine a proper rate for water service. As a result of his study and recommendation a rate was established satisfactory to both parties. Here was the case of a man who could serve either party to a controversy and satisfy both. Why was it? Because he was fair and square, and both sides knew that his decision would be rendered purely on the basis of what was right. And, of course, there was no question as to his technical ability.

"Several years ago, while in a large hospital, he watched a patient in the use of a drinking fountain. What he saw caused him to watch others, there and elsewhere. As he turned over in his mind these facts of observation he became convinced that elements of danger existed in the usual type of vertical jet drinking fountain because of lip contact with the jet spindle and the possibility of pathogenic bacteria remaining in the water column, much as a ball can be balanced on a vertical column of air. Examination proved his fears to be well founded, and inquiry through State boards of health revealed the fact that suspicion was arising elsewhere. He gathered all the information he could and presented a paper before the Iowa Section of the American Water Works Association, recommending the appointment of a committee to thoroughly investigate this problem of public health. The committee was appointed, Mr. Dunlap was made Chairman, served for three years, made a thorough investigation, enlisted the support of various agencies, gathered all available information and substantiating evidence on the subject, and submitted constructive recommendations for the adoption of a specific principle in drinking fountain design which would eliminate all possibility of disease transmission in the manner mentioned. These recommendations were published in the *Proceedings* of the American Water Works Association, have been commented on very favorably by sanitary authorities, and will result in much improvement in this important matter.

"How many of us watching the apparently little things and after turning them over in our minds assign to them a proper valuation? That was what John Dunlap was doing all the time. His mind carried over from a demonstration or experiment in physics to the drinking fountain jet of water, and he saw there an element of danger.

"Many of the substantial things which have been accomplished in this State could be traced back to their initiation in the mind of John Dunlap. He was an inspiring and a directing genius, but in his characteristic quiet way. In many instances others received the credit that should have gone to him. We have been wont to claim credit for the passage of the Engineers' Registration Law, but we should not forget that John Dunlap was the quiet director of our efforts and to him should go a very large measure of credit for the creation of a Board of Engineering Examiners which will justify itself in the years to come after the trying early years have passed.

"John Dunlap was an example of the best type of engineer. He was active and able in the profession; he gave to his fellow engineers the benefit of his experience and observations through papers and otherwise; he entered heart and soul into the activities of those technical societies with which logically he should affiliate; he was a worker in public undertakings for the benefit of society, whether local, State, or National; he took active part in all the worth-while local affairs of his community; he was a pillar in his church; and, above all, he was loyal and true to his home and his family.

"What more can we say of John Dunlap than that he was a real man, a real engineer, and a real friend? He left us while yet a young man and his work but nicely started, yet the place he filled will long be vacant. The lesson he left us is this: Engineering provides an excellent opportunity for service of the highest type, if we but have the point of view. May we not all make honest endeavor to get more nearly the point of view which John Dunlap had, to the end that we may make our profession nobler and worthier of universal respect."

Dean Raymond spoke of Mr. Dunlap's work as a teacher and a citizen of Iowa City, saying:

"Mr. Dunlap was one of those teachers who are keenly alive to the desirability of improving curricula and teaching methods, particularly to the desirability of so improving them as better to develop the man in the student, by giving him breadth of vision and wholesome example. Clean in his living before these students, conscientious in his preparation for his appearances before them in the classroom, sympathetic, generous, and sound with his counsel as they came to him with their personal problems, or their problems of class, or general student activities, interested in their sports as well as in their work, Mr. Dunlap was an invaluable influence for good in the college.

"In the University organization, too, he was recognized to be one of the most useful of men, holding positions on the more important committees. Uniformly courteous even when differing radically with his colleagues, always finding good in the weakest of human beings, the right arm of his pastor in the work of his church, leader in the social service work of the community, John Dunlap unconsciously exerted an influence in University and community life that rarely is contributed by any one man. Not soon will he or his work in Iowa City and the University be forgotten."

In the brief period that Mr. Dunlap was permitted to serve the American Society of Civil Engineers as its Secretary, he made a most remarkably favorable impression on the Board of Direction to which he was immediately responsible, and gave evidence of a real vision of future Society usefulness, to the realization of which he was giving of the best that he had.

One who was familiar with his work as Secretary has this to say of him:

"Fresh from the activities of university life, enthusiastic for his new work, Mr. Dunlap threw himself into the service of the Society. There he found an outlet for his utmost efforts. For two short years he gave all his energies to the many-sided tasks that confront the Secretary of a National engineering organization. Wherever it extended, the Society work felt the impetus of his dynamic energy. This resulted in improvements in the internal machinery of the Soceity, in the quickening of the Society in its relations with its members, and in the furthering of its external co-operation with other activities.

"It was at Mr. Dunlap's suggestion that comprehensive sub-committees of the Society's Committee on Technical Activities and Publications were established. He felt that thereby this important technical work would gain the interest, the counsel, and the assistance of a widespread membership. On his recommendation the Board of Direction adopted changes in the method of handling membership applications. These improvements included the establishment of local membership committees to secure more effective assistance in determining the qualifications of applicants, the revision of the application form, and the classification of applicants for action by the Board, whereby the unquestioned cases were approved more expeditiously and the intermediate to doubtful cases could thus receive a more careful attention.

"Following Mr. Dunlap's suggestion, a questionnaire was sent to members of the Society to learn their special engineering interests. When these forms were catalogued they served to indicate the direction and extent of the major activities of the membership and to provide useful lists of individuals to serve the Society in their particular fields.

"Perhaps his external relations—with Local Sections, Student Chapters, and individual members—were even more successful in their results. For example, he visited every Local Section; in some cases he was enthusiastically urged to return for the second and even the third conference. Similarly, he talked to the Student Chapters, with which his long teaching experience stood him in good stead. Mr. Dunlap's theory was that the administrative office of the Society, its individual members, and the Local Sections, could best co-operate as they knew each other better and appreciated the other's problems; that he could take the Society to the Local Sections and, in return, bring back timely suggestions for its improvement, thus cementing the two parts of the work through mutual understanding and interchange of ideas. These visits had the further effect of strengthening the Society before the public through the favorable presentation of its ideals which he made as an effective speaker.

"Finally, Mr. Dunlap's efforts were marked by a cordiality in the Society's relations with other organizations. Early in his official work he evinced a warm interest in the efforts of the Engineering Societies' Employment Service. He regarded this as a most potent force for assisting engineers and for improving the standards of engineering employment. At a critical period in the existence of this activity, he won over for it the Society's continued support. Even before his death the recovery of the Employment Service had vindicated his judgment.

"Another co-operative effort from which Mr. Dunlap expected great things and to which he gave much study was the Joint Conference Committee (comprising the Presidents and Secretaries of the Founder Societies) formed to consider important matters of common interest to the four National Societies. During its first year, Mr. Dunlap, as Secretary of this Committee, was active in formulating policies under which it has since continued to prosper.

"In still another direction—engineering education—he played an important part. From its inception he was closely connected with the investigation of engineering colleges and curricula carried on by the Society for the Promo-

tion of Engineering Education: First, as a member of the Development Committee of that Society, which formulated the project; then, as a member of its Board of Investigation and Co-ordination, which superseded in the actual conduct of the work; and, finally, as an *ex officio* representative of the American Society of Civil Engineers on the Board of Educational Councillors, a committee of the National Engineering Societies co-operating in the investigation. Thus, in his dual capacity, he was of invaluable help to the work, forming a strong point of official contact between engineering teaching and engineering practice.

"In all Mr. Dunlap's official relations—with his confrères at Headquarters, with other officers, with members or students, with outside organizations—his motto was 'Service'. Whether it was a fundamental policy of the Society or a courtesy to a single distant member, the matter received his closest attention. He gave his all, without stint; this service was not only the last, but the greatest of his life—great in its conceptions and great in its accomplishments."

On August 1, 1924, the Executive Committee of the American Society of Civil Engineers adopted resolutions, as follows:

"*Whereas,* it has pleased Almighty God to take from us our beloved Secretary,

John Hoffman Dunlap

"*Be It Resolved:* That the President and the Executive Committee, in behalf of the Board of Direction and the members of the American Society of Civil Engineers, express their sense of the great loss which the Society and the Engineering Profession of the United States has suffered by the death of our Secretary, who by his labors in our behalf; by his faithfulness, efficiency and courtesy; by his advocacy of the highest ideals for the advancement of the Profession; by his unselfish efforts to serve every member of the Society; and by the example he set us as a Christian citizen, has made a lasting impression in our hearts.

"*Be It Further Resolved:* That our heartfelt sympathy be extended to his family, that his family be furnished with a copy of this resolution, and that it be spread upon our records, printed in the *Proceedings* and *Transactions,* and that copies be furnished to the technical press and to other Engineering Societies."

It seems to be fitting to close this memoir with a repetition of the inspiring words of Mr. Dunlap spoken as he retired from the Presidency of the Iowa Engineering Society and quoted by Professor Nichols in his address before the Memorial Session of the Iowa Section of the American Society of Civil Engineers:

"'Shall we not each of us dedicate ourselves, our lives, our training, our property, all that we have and are, in order that we may serve as good citizens of a noble country, and of a world growing steadily better, because it is an engineer's world in which each engineer is playing his part, with a firm and steadfast faith that the soul of civilization is not meat and raiment, but life and spirit.'"

HENRY FOX, M. Am. Soc. C. E.*

Died June 8, 1925.

Henry Fox was born at Troy, N. Y., on September 12, 1864, and was educated at the Baraboo, Wis., High School.

* Memoir prepared by the Misses Pearl C. and Esther Fox, Baltimore, Md.

He entered the service of the Chicago and Northwestern Railway Company in the spring of 1885, as Rodman, and remained in that position during the construction of the Northern Illinois Railway. In 1886, he was employed as Levelman, on the survey and construction of the Maple River Railway, at Onawa, Iowa. During 1887, he was in camp in the woods of Northern Michigan, while in charge of a division of the Iron River Railway then being built between Iron River and Watersmeet.

In 1889, Mr. Fox entered the University of Wisconsin as a Special Student, changing in the Sophomore year to the Mechanical Engineering Department, and was graduated in 1892 with the degree of B. M. E. During his summer vacations he was employed by the Chicago and Northwestern Railway Company, on general maintenance-of-way work. As soon as he left the University, he was engaged as Inspector by the Chicago and Northwestern Railway Company, during the erection of the Hall System of automatic block signals on the Galena and Wisconsin Divisions. In 1893, he became Mechanical Draftsman at St. Joseph and Hermann, Mo., with the Missouri River Commission.

In 1895, Mr. Fox was employed on the Illinois and Mississippi Canal (Hennepin Canal) and the Chicago and Northwestern Railway, and in 1896, served as Resident Engineer on railway construction for that Company at Gillett, Wis. From 1896 to 1897, he served as Assistant Engineer for the Railway Company at Manitowoc, Wis., in charge of the construction of tracks, buildings, docks, and car-ferry slips. From 1898 to 1900, Mr. Fox was employed as U. S. Inspector for the Illinois and Mississippi Canal, in charge of the construction work of the Second Division, Eastern Section.

From 1900 to 1902, he held the position of Locating Engineer on the Choctaw, Oklahoma and Gulf Railroad, at Hartshorne, Okla.; and from 1902 to 1905 that of Junior Engineer with the U. S. War Department, on field work, in charge of secondary triangulation on seven deep waterway surveys on the Illinois and Des Plaines Rivers from Lockport to Grafton, Ill., a distance approximating 300 miles. On the completion of this work, he was transferred from Chicago, Ill., to Baltimore, Md.

From 1905 to 1909, Mr. Fox served as Assistant Engineer in the U. S. Engineer's Office at Baltimore in local charge of river and harbor improvement and fortifications, as well as ship channels in the Chesapeake and Patapsco Rivers, leading to Baltimore. From 1909 to 1915, he was connected with the Maryland Dredging and Contracting Company (now the Arundel Corporation of Baltimore), as Superintendent and Engineer of general river and harbor improvement work along the Atlantic seaboard, and was engaged in construction work at Buzzards Bay and on the Cape Cod Canal. He also supervised the contract work on the New York State Barge Canal, and on bridges, locks, and dams on the Mohawk River.

From 1916 to 1917, he held the position of U. S. Assistant Engineer at Little Rock, Ark., in charge of field parties for the survey of the Arkansas River, from Little Rock to the mouth of the river, a distance of approximately 150 miles. In 1917, he was relieved from duty and ordered into active mili-

tary service as Captain of Engineers and served during the World War until 1919.

During the latter part of 1919 and part of 1920, Captain Fox was employed with the Baltimore and Ohio Railroad as Assistant Engineer to the District Engineer of Construction, on new tracks, bridges, and engine terminal facilities, such as roundhouses and machine shops, at Grafton, Fairmont, and Clarksburg, W. Va.

In 1920, he returned to the U. S. Engineer Department as Assistant Engineer on construction, Division No. 1, Wilson Dam, Muscle Shoals Project, on the Tennessee River at Florence, Ala. When this work was discontinued on May 1, 1921, he entered the State Highway Department of Florida at Tallahassee, as Location and Construction Engineer, and built concrete bridges and 20 miles of State Highway No. 8.

During 1922, Captain Fox was employed as Engineer with Trammell and Holway, Engineers, on the Spavinaw Water Project for the City of Tulsa, Okla., and located 55 miles of 54 and 60-in. conduit lines at Spavinaw Dam and Tiawah Tunnel. In 1923, he accepted a position with the Walbridge-Aldinger Company, Contractors for the Spavinaw Water Project, at Tulsa, for which Company he constructed 55 miles of standard-gauge railway.

During 1924, he entered the State Highway Department of Oklahoma as a Federal Engineer in charge of field parties in the northern part of the State, which position he held at the time of his sudden illness and death on June 8, 1925.

On October 18, 1893, he was married to Pearl Irwin Clark, of St. Louis, Mo. He is survived by a son, Henry Clark Fox, a student in Civil Engineering at the University of Maryland, and two daughters, Pearl C. and Esther Fox, of Baltimore.

Captain Fox had been a member of the Western Society of Engineers since 1900, and of the American Association of Engineers (Washington, D. C., Chapter), since 1921. He was also a member of Lodge 34 A. F. and A. M.; of Lodge 49, of Baraboo, Wis.; a Thirty-second Degree Mason; and a Member of Zamora Temple and Alabama Consistory, of Birmingham, Ala.

Captain Fox was elected a Member of the American Society of Civil Engineers on July 1, 1909.

MILTON HARVEY FREEMAN, M. Am. Soc. C. E.*

Died March 24, 1925.

Milton Harvey Freeman was born at Crary Mills, St. Lawrence County, N. Y., on October 12, 1871. He was the son of the late Noel O. Freeman whose family came from Vermont, having settled there in Colonial days. His mother was Mary E. Harvey, whose family on the maternal side settled at Gill, near Northfield, Mass.

* Memoir prepared by James F. Sanborn and Ole Singstad, Members, Am. Soc. C. E.

Mr. Freeman was graduated from the Potsdam Normal School, at Potsdam, N. Y., in June, 1895, and for the next two years, taught in district schools in New York State. In September, 1897, he became principal of the Heuvelton Union Free School at Heuvelton, N. Y., where he remained until June, 1899. In the fall of that year, he entered the University of Michigan, with the idea of specializing in mathematics and physics for the purpose of teaching these subjects. While at college, his interests were directed toward engineering as a profession, and he was graduated from the University in June, 1903, with the degree of Bachelor of Science in Civil Engineering.

Immediately after his graduation, Mr. Freeman was employed by the Michigan Central Railroad Company as Assistant Engineer in charge of party, locating branches for logging, double-tracking the main line, laying out freight yards, and maintenance-of-way work. He continued in this position until September, 1905, when he returned to the East and was employed by the Pennsylvania Tunnel and Terminal Railroad Company on the East River Section of the Pennsylvania Tunnel, where he remained as Inspector of shield tunnels until 1907. From 1907 until May, 1909, he served as Assistant Engineer in charge of a shift on shield-driving, building iron lining, and placing the concrete lining in four tubes westward from the Long Island City Shaft.

In May, 1909, Mr. Freeman accepted a position with the Board of Water Supply of New York as Inspector in rock tunnel and in charge of a shaft field party at the Rondout Siphon, where he continued until December of that year, when he was made Assistant Engineer in the Designing Department on studies of reinforced concrete. In March, 1910, he became Assistant Engineer-Chief Inspector on placing concrete tunnel lining. He was engaged in this work until March, 1911, when he became Section Engineer at the Rondout Siphon in charge of concreting and grouting the tunnel lining, which included tests and experimental work in grouting porous rock. He continued at this work until April, 1914, when he was made Section Engineer on the Narrows Siphon in charge of experimental work on the flexible water-tight joint for the 36-in. cast-iron pipe.

In September, 1914, Mr. Freeman entered the employ of the New York State Public Service Commission for the First District as Resident Engineer on the Brooklyn side of the Old Slip-Clark Street Tunnel, from mid-river to the Brooklyn Approach. These were shield-driven tunnels under the East River and the streets of Brooklyn and were to be included in the Rapid Transit Railroad System of New York. In December, 1916, he became Resident Engineer in charge of the entire River Section and Manhattan Approach of the 60th Street, New York-North Jane Street, Long Island City Tunnels, where he remained until the completion of excavation in October, 1918. He was then made Resident Engineer on the Manhattan shore of the 14th Street Eastern-North 7th Street Brooklyn Tunnel, and was employed on this work until June 30, 1919. During the latter period, he was also busy devising grouting methods and was in charge of the field work of stopping leakage at the Canal Street Station of the Brooklyn Rapid Transit Company in New York.

On July 1, 1919, he was appointed Resident Engineer to the New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission under the late Clifford M. Holland, M. Am. Soc. C. E., Chief Engineer. He served with Mr. Holland and his associates in the work of investigation and preliminary design, and on January 1, 1921, became Division Engineer. On July 1, 1921, he was promoted to the position of Engineer of Construction, and remained in active charge of the construction work until the death of Mr. Holland, whom he succeeded as Chief Engineer on December 1, 1924.

The construction of The Holland Tunnel, formerly known as the Hudson River Vehicular Tunnel, is a monument to the high talent and painstaking ability of Mr. Freeman as a subaqueous tunnel engineer. Under Mr. Holland, Mr. Freeman had personal charge of construction on this pioneer project which far exceeds in proportions any tunnel built under similar conditions. The successful driving of five shields of a diameter of 30 ft. 4 in. and one of 31 ft. in itself suffices to mark him as a great engineer. The difficulties encountered in shield-driving, together with the problems of sinking five large shafts, one—the New York River Ventilation Shaft—surpassing in size any similar construction; the water-tightness of these enormous tubes; and the intricate concrete work on the inner lining, were problems that called for his best efforts. The success attained in these various branches of the work stand out as distinct accomplishments of high engineering ability.

Weakened by close application for many years to the numberless details of the work under his charge, his career was prematurely ended. He died at his home in Valhalla, N. Y., on March 24, 1925, of pneumonia after an illness of eight days.

Mr. Freeman was noted for a thoroughness in his knowledge of every detail and for his integrity of purpose and judgment in the solution of the intricate problems which were his responsibility. His devotion to his professional activities was intense and no labor was too great for him to give to the correct solution of whatever problem might be presented to him. He had a fine sense of justice and was noted for his consideration in never doing anything to hurt the feelings of his associates or subordinates. He had high ideals and was fearless in carrying through his purpose with standards of action that marked him among men. His influence was keenly felt by those who came in contact with him, and his death leaves with them a sense of irreparable loss. With his ability was an unusual modesty which always marked his relations with others. One of the outstanding accomplishments of his professional career was the success that attended his work in stopping the leakage at the Canal Street Station of New York's Rapid Transit System. He prepared a paper on this subject, which he read before the Brooklyn Engineers' Club on April 15, 1920, and for which he was awarded the Alfred T. White Prize by the Club.

In 1917, Mr. Freeman was married to Gertrude Vandermark, of Stone Ridge, N. Y., who survives him.

The New York State Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission, of which Commissions Mr. Free-

man was the Chief Engineer, adopted unanimously by a rising vote the following resolution as a mark of respect to the memory of their late Chief Engineer:

"Again the New York and the New Jersey Bridge and Tunnel Commissions met in special session with a vacancy in the office of Chief Engineer. Milton H. Freeman, Chief Engineer to the Commissions, answered the final summons on Tuesday, March twenty-fourth, nineteen hundred and twenty-five, midway in the fifty-fifth year of an active, useful life. Appointed Chief Engineer December 1, 1924, to succeed the late Clifford M. Holland, he brought to the office eminent qualifications of education, trained judgment and wide practical experience. It may be said that he gave himself too unsparingly to the building of the Holland Tunnel. His service was intensive and untiring, his ways so quiet and unobtrusive, his words so considerate and his manner so courteous as to inspire respect and affection, making every member of our Commissions sensible of a personal loss and of the loss to the great undertaking in which we are engaged."

He was a member of the Brooklyn Engineers' Club and the University of Michigan Club of New York.

Mr. Freeman was elected an Associate Member of the American Society of Civil Engineers on July 1, 1909, and a Member on September 9, 1919. He served as a representative of the Society on the Joint Committee on Specifications for Concrete and Reinforced Concrete.

FRANK MILLER, M. Am. Soc. C. E.*

Died May 29, 1923.

Frank Miller the fourth son of Matthew and Sara Anne (Thompson) Miller, of Manchester and Sale, Cheshire, England, was born in Manchester, England, on June 23, 1862. After a preparatory education in private schools in Sale and Southport, he went to Germany and continued his education, attending schools in Lahnestein, Düsseldorf, and Dresden, where he was a student at the Polytechnicum. During the summer vacations he traveled with a private tutor and studied art in many cities of Europe.

On his return to England, his father wanted him to enter the British Army, or to join him in his business of cotton manufacturing, as he had large interests in Lancashire, but the boy preferred to follow his own inclination in the choice of a profession and, finally, decided to enter Owen's College, Victoria University, England, to continue his study of engineering. He was for some time a lecturer at Owen's College.

For one year Mr. Miller served as Machinist and Draftsman at Appleby Brothers' Bridge and Crane Works, near London. From 1883 to 1885, he was Assistant in the office of Sir John Fowler and the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., Consulting Engineers, London, making plans for bridges, masonry construction, railroads, and details for the Forth Bridge in Scotland. From 1885 to 1888, he held the position of Assistant Engineer of the Lancashire and Yorkshire Railway Company, in charge

* Memoir prepared by John B. Humphreys, Esq., Paterson, N. J.

of design and construction for four-tracking 10 miles of a difficult section of the road, including bridges, station buildings, retaining walls, etc. He also made marine surveys for deepening and improving the channel and harbor at Fleetwood, England.

In 1888, Mr. Miller went to Montreal, Que., Canada, to gain wider experience and to find a larger field in engineering work, and until 1889, he was connected with the Grand Trunk Railway Company of Canada, making surveys for railroad extensions. From 1889 to 1891, he served as Draftsman, Designer, and Estimator for Riter and Conley, Steel Works, Pittsburgh, Pa., and from 1891 to 1892, he was engaged in designing new structures for the Brooklyn Elevated Railroad under the late O. F. Nichols, M. Am. Soc. C. E., Chief Engineer. From 1892 to 1895, he was with Cooper, Hewitt and Company, at Trenton, N. J., at the plant of the New Jersey Steel and Iron Company, in charge of draftsmen, designing bridges, buildings, and structural steel work of all kinds, and making estimates.

From 1895 to 1903, Mr. Miller was in private practice as Consulting and Constructing Engineer and as a member of the Structural Engineering Company, of New York, N. Y. During this time a large number of difficult engineering works were designed and constructed successfully, including the plant of the Central Lard Company at Jersey City, N. J., a large building filled with heavy oil tanks on every floor. This building, the cost of which was about $250 000, was constructed on a swamp. The Passaic Iron Foundry was also built under the supervision of this Company, as well as the steel work of numerous office, loft, and mill buildings and bridges, including their foundations. The Long Beach, N. Y., Draw-Bridge and various pile protection works were also designed and their construction superintended by the Company. In 1902, Mr. Miller designed, estimated, and superintended the construction of the floating derrick of 120 tons capacity for the United States Navy Yard at Portsmouth, N. H.

From 1903 to June, 1909. he was Chief Engineer of the Snare and Triest Company, Contracting Engineers, during which period he designed and superintended the construction of the coffer-dams for the building of the foundations for, and also superintended the erection of the steel work of, the highway draw-span of the Flushing Bridge, Flushing, N. Y., built at a cost approximating $280 000. Mr. Miller designed and estimated the cost of large fireproof warehouses, and railroad locomotive and repair shops, built for the United Railways, at Havana, Cuba, at a cost of $500 000. He also designed and estimated the cost of three concrete and steel ocean piers, built for the United Railways of Havana and the Cuban Central Railway, at Havana and Cienfuegos, Cuba, as well as numerous bridges (for railroads), and coal-handling plants. He planned the method of building the Manhattan Subway Terminal of the Williamsburgh Bridge, New York, N. Y., the contract price of which was about $1 100 000; he also designed and superintended the shoring and underpinning of the adjoining buildings, as well as the shoring of the street railway tracks and temporary roadways.

Mr. Miller designed the plant and method of erecting the steel work of the approaches of the Queensborough Bridge at 59th Street, New York, and Long Island City, N. Y., over the East River. He designed the coffer-dams, pile and concrete construction of the tunnel connecting the Delaware, Lackawanna and Western Railway Station at Hoboken, N. J., with the Hudson and Manhattan Tunnel, and arranged the method of construction. He also designed the coffer-dams, foundations, and steel work for the Perth Amboy Bridge across Raritan Bay and superintended the construction. He designed, estimated and superintended the construction of sixteen large floating derricks for The Merritt and Chapman Derrick and Wrecking Company and The M. P. Smith and Sons Company, both of New York. These derricks had a lifting capacity of from 25 to 150 tons, and were built at a total cost of about $500 000. This work, executed for The Snare and Triest Company, represents some of the large contracts, but, in addition, Mr. Miller had charge of all engineering matters affecting designs and erection, and of a large part of the estimates.

He subsequently entered into partnership under the firm name of Long and Miller, Consulting and Contracting Engineers, New York. While a member of that firm, he was interested in the development of Larchmont Gardens, reconstruction work at Fort Slocum, and the widening of a part of Fifth Avenue, New York, and was also engaged on many other pieces of engineering and construction work in and around that city. The firm of Long and Miller dissolved partnership in 1915, and Mr. Miller continued the practice of his profession alone, as a Consulting Engineer with an office in New York.

At the outbreak of the World War, he offered his services to both the United States and Great Britain, but was not accepted on account of his health. He had hoped that his experiences as an engineer would be of service to the Allied Forces.

For many years Mr. Miller served as Consulting Engineer for Messrs. Merritt and Chapman, and was associated with that Company on several of its large contracts, notably the laying of the Narrows Siphon. It was Mr. Miller who raised the American liner *St. Paul* after she had turned over at her pier at the foot of West Twenty-second Street, New York, and had lain fast in the mud for some time.

In April, 1920, he visited the West Indies, in order to make plans and specifications for harbor works at East Harbour in the Turks and Caicos Islands group.

In June, 1920, he became seriously ill, but, after being in a critical condition, he finally recovered sufficiently to resume his business. From time to time, he suffered greatly, but never allowed his health to deter him, to curb his ambition, or to handicap him in the execution of any piece of work he had in hand.

At different times, Mr. Miller did work for and in connection with the Keystone Cement and Fireproofing Company. He was also interested in the development of Belwood Park, a part of the Hewitt Estate, near Newark, N. J.

The last work that Mr. Miller undertook and executed was the design for the great dome on the new Roman Catholic College, Marywood, at Scranton, Pa., which is considered a very beautiful piece of work, the second or third largest dome in the United States, and, as one of his business associates said, a "lasting tribute to his wonderful talent for design, and to his memory". He had the gratification, in the last few weeks of his life, of knowing that it had been successfully erected.

His profession was to him an art, and his genius comprised a diversified talent for design, but he also inherited from his Scotch ancestry his strongly practical characteristics.

In the autumn of 1922, Mr. Miller suffered a severe recurrence of his previous illness, and notwithstanding a very brave effort on his part to overcome his malady, he never regained his normal strength, although striving to the last to continue active work. He died on May 29, 1923, after two weeks of acute illness.

In 1889, Mr. Miller was married to Ada Elizabeth Sawyer, the only daughter of Thomas Spencer Sawyer, of Deeplish, Rochdale, England, who, with one daughter, Dorothy Ethel, survives him.

In social life Mr. Miller was popular and beloved by many. He had a high sense of humor, not boisterously exhibited, but when greatly amused, a sort of concealed laughter permeated his entire body. His life was a kindly and a useful one.

He was a member of the Canadian Club, and the Engineers Club of New York. In religion, he was a member of the Protestant Episcopal Church, and usually attended a daily service at one of the city churches. He was an ardent lover of the best in music, this taste having been fostered and cultivated during his long residence in Germany.

Mr. Miller was elected a Member of the American Society of Civil Engineers on October 5, 1904.

CHARLES OSCAR POOLE, M. Am. Soc. C. E.*

Died April 2, 1925.

Charles Oscar Poole, the son of Reuben and Mary Agnes (Gorace) Poole, was born at Saulsbury, Mass., on June 17, 1859. His father was a Mechanical Engineer and came to the United States from Yorkshire, England.

When Mr. Poole was ten years of age, the family moved to San Francisco, Calif. He attended the public schools there and later took special studies in Mechanical Engineering, which profession he intended to follow.

From 1880 to 1884, he served as Machinist and Blacksmith for the Oregon Improvement Company in the State of Washington, leaving to work for four years with the Newcastle Coal Mine near Seattle, as Steam Engineer. In 1888, he returned to the Oregon Improvement Company as Master Mechanic of the Seattle Division, which position he filled for three years.

* Memoir prepared by E. J. Waugh and F. O. Dolson, Members, Am. Soc. C. E.

In 1891, Mr. Poole returned to San Francisco as First Operator and, later, as Foreman of the Dynamo Room of the California Electric Light Company of San Francisco. In 1894, he was made General Foreman of Shop and Maintenance for the Company, which position he held until, after a consolidation of interests in 1897, he was made General Superintendent of the Electrical Department of what was then known as The Edison Light and Power Company of San Francisco. During the next three years, he was in charge of the management of the entire electric light and power business of San Francisco, except the street railways.

In 1900, he resigned from this position to become General Superintendent and Assistant Manager of the Standard Electric Company of California. In this capacity, Mr. Poole had entire charge of the Construction and Operating Departments and under his direction the early hydro-electric development at Electra was undertaken and the first high-tension transmission line was constructed into San Francisco. During this time, Mr. Poole was interested, also, in the United Gas and Electrical Company which held the electric and gas supply franchise in San José, Calif., and much of the business of this Company was under his supervision as Manager and Supervising Engineer.

In 1902, he accepted a position as Sales Engineer for the Stanley Electric Company of Pittsfield, Mass., and had his headquarters with the Hendrie and Bolthoff Manufacturing and Supply Company, at Denver, Colo. Through this connection, he became acquainted with a Denver group of capitalists who, in 1904, began to construct a hydro-electric power development on Bishop Creek, California, with a power transmission line to Tonopah, Nev. From 1904 to 1905, Mr. Poole was Designing Engineer, with offices in Denver, for the Company, then known as the Nevada Power Mining and Milling Company, and, from 1905 to 1907, he served as Assistant General Manager with offices in Goldfield, Nev.

In 1907, The Nevada California Power Company was organized and from August, 1907, until October, 1923, Mr. Poole was its Chief Engineer. Several allied companies were afterward formed, of which he was also Chief Engineer, notably The Southern Sierras Power Company, The Sierras Construction Company, the Hydro-Electric Company, the Pacific Power Company, and many others.

In 1910, Mr. Poole formed a partnership with Mr. R. G. Manifold, the firm being known as Manifold and Poole, Consulting Engineers. They were the joint authors of "Straight Line Engineering Diagrams". The firm designed the steel tower transmission line of The Southern Sierras Power Company, a pioneer in length and voltage. It is 237 miles long and operates at 87 000 volts from Bishop Creek, in Inyo County, to San Bernardino, Calif.

As Chief Engineer in charge of design, construction, and, for many years, of operation, Mr. Poole was, in a great measure, responsible for the growth and development of the systems of The Nevada California Power Company, The Southern Sierras Power Company and Affiliated Companies, and for the development of the territory which these Companies serve.

In October, 1923, he resigned as Chief Engineer, moved to Los Angeles, Calif., and resumed general practice as Consulting Engineer with Mr.

Manifold, as Manifold and Poole. He continued as Consulting Engineer for The Nevada California Power Company group until his death.

Mr. Poole was married in 1913 to Mrs. Cora Weston, of Los Angeles, who survives him. A son, Walter, in the United States Marine Corps, at Mare Island, Calif., and a daughter, Mrs. Erma Lorenson, of Oakland, Calif., children by a former marriage, are also living.

His work covered a period when great strides were made in the electrical industry, and he did as much as any one man in California to extend and simplify the electric and especially the hydro-electric industry of the State. He was foremost in the development of outdoor sub-stations, and long, high-voltage transmission.

Mr. Poole had a fine mind, a faculty of keen observation, and an exceptional memory. He was a clear and forceful conversationalist and speaker, with a ready fund of anecdote and quick wit. He was a truly strong, constructive personality. There were two outstanding characteristics that set him apart from ordinary men. One was a driving energy of mind and body that would not be satisfied with anything partly known or partly done. He typified the dynamos with which he worked and understood. The other characteristic was loyalty and steadfastness. Once having started on a course, he did not waver and he was ever a loyal friend. He will be greatly missed in the Engineering Profession both for his work and for his good fellowship and social qualities.

At its meeting of August 12, 1925, the following resolution was adopted by the Los Angeles Section of the Society of which Mr. Poole was a member:

"Charles Oscar Poole, M. Am. Soc. C. E., a man from our midst, died on April 2, 1925. We, his friends, can but offer to his memory the highest possible tribute for a life, highly successful as the world terms success, but more, invaluable for its contribution to the advancement of his chosen profession, and for its personal inspiration, which we who knew him well so greatly appreciate and will so greatly miss.

"Since a young man Charles Oscar Poole was closely identified with electrical and hydro-electrical development. His work covered a period when great strides were made in this field, and he did as much as any one man in California to extend and simplify the electrical, especially the hydro-electrical, industry of the State. He was a leader in the development of outdoor sub-stations, and of transmission over long distances. To his efforts can be credited the first high-tension transmission line into the City of San Francisco, where he had for some years control of the entire electric light and power business of that City, excepting only the street railways. The steel tower transmission line of the Southern Sierras Power Company, a pioneer of length and voltage, is a monument to his efforts.

"Courage, untiring energy, vision and love of pioneering work, and loyalty to high ideals were his, and it is these traits of character, as well as his accomplishments, which lead us, his fellows in endeavor, to tender our respect to a man whose memory can only lead to greater progress, such progress as was so strongly the motive of his life."

Mr. Poole was a Fellow of the American Institute of Electrical Engineers and was elected a Member of the American Society of Civil Engineers on October 21, 1924.

AMOS SCHAEFFER, M. Am. Soc. C. E.*

Died October 3, 1924.

Amos Schaeffer, the son of Augustus and Amanda (Leibensberger) Schaeffer, was born on a farm at Fleetwood, Berks County, Pa., on February 17, 1867. He was of German ancestry, and of the people who populated a large part of the State of Pennsylvania and made it the flourishing and prosperous section that it is to-day.

His early education was obtained in the public schools, after which he was graduated from the Kutztown, Pa., Normal School. Following a few years of teaching, he entered Franklin and Marshall College at Lancaster, Pa., from which he was graduated in 1889 with the degree of Bachelor of Arts and finished with a year at Sheffield Scientific School, Yale University, from which he obtained the degree of Bachelor of Philosophy in 1890.

Following his graduation, Mr. Schaeffer was employed by the Lehigh Valley Railroad Company on surveys and field work. In April, 1892, he entered the service of Rodgers and Clement, Contractors for the Niagara Falls Power Tunnel, in charge of portal masonry and stone-cutting for the tunnel, which is about 1½ miles long, and is the tail-race for the water that enters the turbines after dropping perpendicularly about 150 ft. at the forebay at Niagara River above the Falls. The tunnel was built under the City of Niagara Falls for the first utilization of the power of the Falls by electricity (1895), and is still in use.

During parts of 1893 and 1894, Mr. Schaeffer was engaged with the United States War Department of Jonesport, Me., in charge of the removal of ledge rock under water in the ship channel and for a short time with the Lehigh Valley Railroad Company on a contour survey for railroad location in Maryland.

In May, 1894, he returned to Niagara Falls, entering the City Engineer's Office as Assistant Engineer, in charge of the design and construction of sewers. He completed the construction of a tunnel trunk sewer in rock; made plans for and superintended the construction of brick and asphalt pavements; designed and constructed the skewbacks and abutments for a 150-ft. span skew steel arch bridge; and located and established street lines and grades. The City of Niagara Falls had grown rapidly in size and population after the introduction of cheap electric power and many new industries had settled there, necessitating additional residential facilities, and during the latter part of his stay, Mr. Schaeffer engaged in private practice on property sub-division and general engineering in connection with the paper and pulp mills which had located there.

In November, 1897, he entered the employ of the City of New York, which, with one short exception, was destined to be his last change. His first work was in the Topographical Bureau, Department of Sewers, Borough of the Bronx. In 1900, on the inception of the work of the Rapid Transit Railroad

* Memoir prepared by William J. Boucher, M. Am. Soc. C. E.

he transferred to the Rapid Transit Commission where he made a specialty of the sewer problems. The construction of this railroad, built as it is, almost entirely below the streets of the City, caused a most elaborate re-design of the sewer system, necessitating many re-arrangements, new construction, and reconstruction of the trunk sewers and lateral systems. On the resignation of Mr. Calvin W. Hendrick in 1905, Mr. Schaeffer was appointed Engineer of Sewers. The careful investigation, study, design, and construction done under his supervision is amply attested by the results after practically twenty-five years' growth of the great city at a rate never anticipated.

In 1909, Mr. Schaeffer resigned from the Rapid Transit Commission and Sewer Division to become Chief Engineer of Gore-Meenan Company, Sub-Contractors for a section of the Catskill Aqueduct on the east side of the Hudson River, near Peekskill, N. Y. Although he worked hard and gave his best thought and efforts to the success of the work, financial difficulties caused a suspension. This, however, was through no fault of Mr. Schaeffer.

In 1910, Mr. Schaeffer was appointed Consulting Engineer, Borough of the Bronx, New York, a new position, reporting to the President of the Borough. Some years later, he was appointed to a similar position—Consulting Engineer, Borough of Manhattan—which he held until his death, serving in the two Boroughs under five different Presidents and three Mayors. While in this position, Mr. Schaeffer's work was of a most varied character and comprised largely, improvements planned for the future betterment of the fast growing Boroughs to accommodate the increasing traffic and population. Vision, forethought, and sound planning were essential, and with his experience and training, he was well equipped to cope with the many problems which were constantly arising. His intimate knowledge of the laws and regulations of municipalities enabled him to frame his recommendations so that they secured speedy adoption. One of the Borough Presidents has expressed the opinion that he was one of the ablest engineers in the City of New York.

Mr. Schaeffer was of a most pleasing and cordial personality; once he had been gained as a friend, he remained such. His contacts were with all classes, and his tact and diplomacy as well as his thorough knowledge of the facts regarding his work and projects enabled him to present the matter in hand in a pleasing and convincing manner.

He was an ardent church worker, having been affiliated with the Fordham Manor Reformed Church, Bronx, New York, of which he was a member and Elder, and a Member of the Classis. In 1924, he was elected one of three Trustees of the Board of Direction of the Reformed Church of America. Shortly after his death, the Consistory of that Church adopted a resolution, which was spread on its minute book and published in the "Monthly Leaflet", part of which reads, as follows:

"Mr. Schaeffer's character was above reproach. His virtues were the virtues of the stoical philosophy as expounded by Gilbert Murray. 'God, the eternal dramatist', says Professor Murray, 'has cast you for some part in his drama, and hands you the rôle. It may turn out that you are cast for a triumphant king; it may be for a slave who dies of torture. What does that matter to the good actor? He can play either part; his only business is to accept the rôle

given him and perform it well. * * * Do you ever find that history praises a man because he was healthy, or long-lived, or because he enjoyed himself a great deal? History never thinks of such things. The thing that lives is a man's goodness, his great deeds, his virtues, or his heroism. * * * The only good for man is to be good. And when Zeno says "good," he means good in the ultimate Day-of-Judgment sense.'

"Mr. Schaeffer accepted his rôle in life in this spirit. He was not concerned about his ease or his pleasure—only his duty. That is why he held his position, unprotected by Civil Service, through all the mutations of political strife. In the City Government he was a Gibraltar that could not be moved by corruption, flattery, or threat. To him the only question concerning any measure was, 'Is it right, and in the public interest?' In the Consistory, these same traits were always in evidence. His goodness was not expressed by words, but by deeds. He had vision and courage, and boldly faced the future even when the outlook was darkest; much of the progress of the church in recent years we owe to him. He served on many important committees, and always with fidelity and distinction. Everything that came under his scrutiny received thorough and conscientious consideration.

"As a friend and neighbor Mr. Schaeffer was kind and true, ready to help in time of need. He was a charming host and had—what is rare with men as serious as he was—a delightful sense of humor. He was a real man, without sham of any kind. 'The reality of things,' says L. P. Jacks, 'is inversely proportional to the noisiness of the self-announcement.' Mr. Schaeffer was as modest as he was efficient. He never sought the spotlight, but was content to play his rôle even when others took the applause.

"The Consistory as a body hereby expresses its regret at the loss of one of its most useful members. As individuals, the members have lost a genuine friend and lovable companion. They offer their profound sympathy to the widow and orphans left by the deceased, and incorporate this expression of their respect and loss into the official record of the church."

Mr. Schaeffer's health began to fail during the summer of 1924; a long vacation seemed to restore him somewhat, but he died of typhoid fever on October 3, 1924. He was married to Florence Harriot Messmore who, with a son and a daughter, survives him.

Mr. Schaeffer was elected an Associate Member of the American Society of Civil Engineers on February 3, 1904, and a Member on February 6, 1906. He was also a Past-President of the Municipal Engineers of the City of New York.

SAMUEL WHINERY, M. Am. Soc. C. E.*

DIED JANUARY 14, 1925.

By the death of Samuel Whinery, at East Orange, N. J., on January 14, 1925, the Society lost one of its oldest and most distinguished members, and the profession lost one of its notable and patriotic figures, one whose devotion to its highest ideals will ever stand as a shining example for its younger members.

Samuel Whinery, the son of Robert W. and Susan (Enlow) Whinery, was born near Salem, Ohio, on November 20, 1845. In 1853, the family moved to

* Memoir prepared by Frederic Molitor, M. Am. Soc. C. E.

a farm in Jennings County, Indiana, where young Samuel attended the public schools. Later, he attended the Friends School near Salem, Ind., and continued his education by two terms at Indiana University.

Between the ages of eighteen and twenty-one, he taught in the county schools. During this period he devoted all his leisure time to systematic study, especially of Civil Engineering. It was in this wise that young Whinery obtained that firm and solid foundation upon which his great scientific and engineering career was built. One must pause to compare the very limited opportunity then afforded for professional education in the Middle West with its present-day colleges and universities equipped with large, highly trained and experienced staffs. Can one gainsay that young Whinery, with only a few textbooks and no teachers, but with a studious inclination, a character formed upon the simple and strong faith of the Friends, the mental solitude of a sparsely settled country, reached a professional goal that is seldom equalled by those fortunate enough to have the advantages of a modern technical education?

About 1868, the Indianapolis and Vincennes Railroad was being promoted to develop the part of Indiana in which Mr. Whinery lived. When the locating party took the field, he joined it, and thus gained his first experience, interest, and apprenticeship in railroad engineering. Within a year (1868), he became Resident Engineer in charge of construction on a 13-mile section of this road.

In 1869, he was appointed Resident Engineer of Construction on the Indianapolis and St. Louis Railroad at Terre Haute, Ind. Following the completion of his residency, he built the railroad shops at Mattoon, Ill., and was then appointed First Assistant Engineer on the location and construction of the Cincinnati, Rockport and Southwestern Railroad in Southern Indiana, on which he was engaged for two years.

In 1873, he accepted a position with the Cincinnati Southern Railroad, notable then as now as being an enterprise built and owned by the City of Cincinnati, Ohio. The survey of this railroad was a distinct advance in the location of railroads. The theory of the economics of railway location was used throughout in the far-sighted policy of its Engineering Staff of which Mr. Whinery was an influential member, the Chief Engineer having been the late L. G. F. Bouscaren, M. Am. Soc. C. E. After the completion of the location, Mr. Whinery had charge of a residency and finally was made Division Engineer in charge of the Chattanooga Division. Thus, at thirty years of age, Samuel Whinery had local charge and the responsibility of the heavy construction of this Division.

Upon the completion of the Cincinnati Southern Railroad, in 1878, Mr. Whinery was induced to accept a position as Civilian Assistant Engineer in charge of the Muscle Shoals Improvement of the Tennessee River. In this position, his experience in charge of railroad construction, his executive ability, and his diplomatic talents were of inestimable value to the officers of the Corps of Engineers, U. S. Army.

In 1881, his services were again sought by railroad companies. He was offered and accepted the charge of the location and construction of the North-

ern Division of the New Orleans and Northeastern Railroad. During the first year of its operation he acted as Superintendent, and it was at this time that the telephone was first used in train dispatching. Mr. Whinery early had seen the many advantages of this new invention, and, consequently, made use of it during the construction stages of the work. When train service was inaugurated, its use was continued as the means of communication between dispatchers and train crews, and he, therefore, was the first to use the telephone for this purpose.

In 1884, the first period of Mr. Whinery's career ended. Upon the location and construction of railroads he had served his apprenticeship in practical engineering and had risen to responsible charge of work. His services and fidelity had been of the greatest aid to his chiefs, his talents had impressed the profession, and his administrative ability had been established. In short, he had acquired that firm basis which qualified him to "go on his own" as a consultant in private practice. For the following three years his work was in the South and he executed many important commissions of varied character, which called for a broad knowledge of engineering. His most important achievement during this period was the design and construction of the Incline Railroad up Lookout Mountain, at Chattanooga, Tenn., rising 1 200 ft. in less than 1 mile.

In 1887, he received a flattering offer from the Warren-Scharff Asphalt Company to direct its installations. For one year, he served as Assistant Manager in charge of its road pavements and engineering work. Here his professional talents found a large field of usefulness in the growing public demands for better streets, and highways. At the end of a year he was appointed Vice-President and General Manager of the Company, which position he held for twelve years (serving as President for the last six months), or until the Company was merged with the "Asphalt Trust".

Notwithstanding the responsibilities of his office with this Company, Mr. Whinery managed to be of service in other fields of engineering. Among other activities, he served as a member of the Engineering Commission to investigate a new water supply for the City of Cincinnati (1896-97), and as President of the Water Commission of Wyoming, a suburb of Cincinnati. In these undertakings his researches and studies again showed his great power of analysis, and his sometimes new and always advanced professional thoughts. His conclusions have long since been proved correct.

In March, 1901, Mr. Whinery opened an office in New York, N. Y. He had not become known to the general public, but his professional standing among his associates had placed him among the leaders of engineering thought and accomplishment. He practiced in New York, with marked success, for twenty-two years, retiring in 1923, with fifty-five years of active professional work to his credit and to the good of his clients and the public.

Among the many important works with which he was identified or directed during this last epoch of his career were the following: Reports (1903) on the transportation problems of New York, and on the extension of the pierhead lines in the North River; Consulting Engineer to the Finance Commission of

Boston, Mass.; Consulting Engineer (1907-08) to the Commission on City Expenditures of Chicago, Ill.; Consulting Engineer (1905) to the President of the Borough of Manhattan, New York; Chairman of the New Jersey Commission to re-appraise railroads and canals; President of the Water Commission of East Orange, N. J.; report on the elimination of railroad grade crossings, at East Orange. He also assisted Gustav Lindenthal, M. Am. Soc. C. E., in research work in connection with some of the details of the famous New York Connecting Railroad Bridge over Hell Gate.

Although beyond the age of military service at the time of the World War, Mr. Whinery offered his services, and was placed by the U. S. Navy Department on the Reserve Officers' Examining Board of the Bureau of Yards and Docks. He spent an arduous term in the civil side of prosecuting the War. His ready and patriotic encouragement was extended to those emergency Engineer Officers who were in active service. The writer cherishes the simple and beautiful words he had from Mr. Whinery, both on leaving for active service in France and on his return. In a word, Mr. Whinery was a patriot and a militant Quaker during the country's experience in the World War.

He was always an advocate of the Engineer taking his part in public affairs, and lending his experience and advice to the proper solution of public questions, where engineering and economics were factors. His was not a passive part; on the contrary he gave liberally of his time and vast experience as evidenced by the fact that for many years he was Chairman of the Committee of Engineering and Sanitation of the Merchants' Association of New York. He also served for several years as President of the Board of Education of East Orange.

His active interest in engineering education in this country, particularly for a broader and better curriculum, led the Universities of Cincinnati, Illinois, and Cornell to invite him to lecture before their engineering students.

Many of the older members of the Society feel that Mr. Whinery's best example to the younger members, and one of his leading contributions to the profession, was expressed in his determined and devoted stand for an established Code of Ethics. The writer believes that it is largely, if not wholly, due to Mr. Whinery that professional conscience was awakened on this subject. At least, it is believed that he was the first to bring the subject before the profession in an explanatory and constructive way.

On his retirement from the Presidency of the Cincinnati Engineers' Club at the Annual Meeting on December 15, 1892, the subject of his address was "Ethics of Civil Engineers", and this exhaustive and constructive paper has been preserved to the profession.* It is worthy to be read by every engineer, because even now, after all the engineering societies have finally adopted Codes, it will be found that few of them have neglected at least some of his early suggestions. In this able address, Mr. Whinery refers to ethics as "being the science of right conduct". Can it be better expressed? Then, again, he said: "Such a code must be built up along the lines of the system of morals, which is universally recognized as the outgrowth and sequence of the Ten Com-

* *Engineering News*, January 26, 1893.

mandments". Could the construction of a code of ethics be builded on a firmer or safer foundation? And, further, he said: "No class of men has the moral right to set up itself a standard of conduct, which, however, advantageous it may be for their selfish ends, shall disregard the rights and courtesies due to those who are not members of their particular calling". Could there be a better interpretation? He said, also: "It is feared that civil engineers are sometimes remiss in that scrupulous regard for the rights and reputations of their professional brethren, which is so carefully observed in other professions". Did he then speak wrongly? Is this statement not as true to-day as it was in 1893?

The duty of the engineer to the public as then expressed by Mr. Whinery is as follows:

"The engineer owes to the public his earnest services as a good citizen in the highest sense that the word implies. He has no right to neglect his share of those political and social duties which fall to the lot of every patriotic citizen. It is a fact frequently remarked that the civil engineer of the better class is not often found in the ranks of the leaders of civic and social life".

This seems unhappily as true to-day as when Mr. Whinery made the statement thirty-two years ago. It was these sentiments that Mr. Whinery spoke among his associates and professional friends. They bore fruit when in June, 1911, the American Institute of Consulting Engineers adopted a Code of Ethics (the first engineering code in the United States, it is believed), prepared by a committee of which the late John F. Wallace, Past-President, Am. Soc. C. E., was Chairman. As a member of this Committee the writer can now testify that Mr. Whinery's address of December, 1892, was one of the guiding factors in its preparation. His influence was doubtless again felt when, on September 2, 1914, the Society adopted a Code of Ethics.

From 1915 to 1922, Mr. Whinery was Chairman of the Committee on Professional Practice and Ethics of the American Institute of Consulting Engineers. During this time the Committee passed upon many cases of the Institute's Code and upon interpretations of it. His work was accomplished so splendidly and carefully that the Institute in 1923, upon his retirement from this important office, named him Chairman Emeritus. He continued to give valuable advice to the Committee, notwithstanding his increasing years. Only a short time before his death he stated that he felt his life work would be incomplete until the Engineering Profession had adopted a Code of Ethics, and it was his hope that engineers would abide by it as the medical and legal professions have done.

Mr. Whinery was the author of "Municipal Public Works" (1903), and of "Specifications for Pavements and Roads". He also contributed largely to the publications of various professional societies and to technical magazines.

He was a Charter Member of the American Institute of Consulting Engineers (1910) and served on its Council in 1916, and from 1919 to 1921, and was also Vice-President for two years, 1920 and 1921. He was also a member of the American Society of Mechanical Engineers, American Institute of Mining and Metallurgical Engineers, American Public Health Association, and the American Academy of Political and Social Science.

Mr. Whinery was married on November 10, 1875, to Elizabeth A. Crawford, daughter of John Crawford, of Somerset, Ky. Besides his wife, he is survived by the following children: Samuel Brent Whinery, an engineer, Charles C. Whinery, Maurice R. Whinery, Andrew J. Whinery, a lawyer, John E. Whinery, and a daughter, Elizabeth Whinery. For the last twenty years of his life, he resided in East Orange, N. J.

It is by the wide variety of his professional work, the detailed research and analyses of his conclusions, and the thoughtful expression of his reports that his professional career is best measured, marking him as outstanding on the long roll of membership in the Society.

This memoir may best be concluded by a statement of Mr. Whinery's worth as a man and as an engineer, contributed by one of the leaders of the profession and one whose gift of expression is so well known and appreciated, William J. Wilgus, M. Am. Soc. C. E.:

"I had for him a rare combination of affection for his lovable qualities and respect for his professional attainments and judicial character. In fact, his sweetness of disposition, ability as an engineer, and devotion to the highest ideals of our calling, marked him as a man out of the ordinary, whose passing from our midst leaves a niche that will be most difficult, if not impossible, to fill. At least, he is enshrined in our memory as one whose example we should all aspire to follow".

Mr. Whinery was elected a Junior of the American Society of Civil Engineers on April 1, 1874, and a Member on May 4, 1881. He also served on the Board of Direction in 1891 and from 1899 to 1901, and as Vice-President from 1892 to 1893.

CHARLES SUMNER WILLIAMSON, M. Am. Soc. C. E.*

Died March 31, 1923.

Charles Sumner Williamson was born at Cleveland, Ohio, on April 12, 1874, the son of Samuel Williamson and Stella E. (Sumner) Williamson. His mother was a direct descendant of Increase Sumner, one of the early Governors of the State of Massachusetts. His father also came from a long line of Colonial families in the United States.

Following his preparatory education in the public schools of Cleveland, Mr. Williamson received his technical and engineering training in the Case School of Applied Science, from which he received the degree of Bachelor of Science in 1895 and that of Civil Engineer in 1898.

From 1894 to 1895, he worked in the office of a local Civil Engineer. From 1896 to 1898, he was in the employ of the King Bridge Company and the Variety Iron Works of Cleveland. Except for this minor work, Mr. Williamson

* Memoir prepared by a Committee consisting of G. H. Hutchinson, *Chairman*, W. H. Hoyt, and J. N. Hatch, Members Am. Soc. C. E., and Messrs. A. M. Merriweather, William S. Monroe, and C. C. Brooks.

was employed in the hoisting and conveying machinery business throughout all his comparatively brief career. One of his earlier engagements was with the Brown Hoisting Machinery Company of Cleveland, with which he served, successively, as Draftsman, Squad Foreman, and Designing and Erecting Engineer, his work having to do with the design and installation of car dumpers for transferring coal from railroad cars to Lake vessels, ore and coal-handling bridges, and other special types of material-handling equipment.

Following this period with the Brown Hoisting Machinery Company, covering the principal part of the time from 1895 to 1904, he was made Contracting Sales Engineer of Heyl and Patterson, Incorporated, of Pittsburgh, Pa., which position he held for seven years. In this capacity Mr. Williamson played an important part in establishing and building up the ore and coal-handling machinery department, which specialized in the development, production, and installation, at the Lake coal and ore docks and at blast furnaces, of ore and coal-handling equipment of the bridge and hoisting tower type.

In 1911, he became associated with the Mead-Morrison Manufacturing Company, Constructing Engineers, of Boston, Mass., as Western Manager, with headquarters at Chicago, Ill., his duties being those of Managing and Contracting Engineer for iron ore and coal-handling machinery. He was later promoted to be Vice-President, still retaining charge of the Western District, and remained with the company in this capacity until he was compelled to retire from active business on account of ill-health. While with the Mead-Morrison Company, Mr. Williamson's activities were not limited to America, but extended to Europe and Australia.

Mr. Williamson was fortunate during his early practical experience, while with the Brown Hoisting Machinery Company, in being brought in contact with Mr. Alex E. Brown, the pioneer in the development and introduction of the mechanical handling of materials, a man of remarkable personality, whose clear vision, inventive genius, indomitable will, and persistent effort had much to do in making possible the world supremacy of the Great Lakes, in the handling and transportation of bulk cargo material.

Although the pioneering work had been done, and much accomplished in the line of the mechanical handling of bulk materials before Mr. Williamson entered the field, he had been intimately associated with the progress made in the previous twenty-five years. His activities covered the entire period of development of the more modern type of material-handling equipment. Various improvements in mechanical detail and in methods of material handling stand to his credit, some of which he protected by letters patent.

Mr. Williamson was a great lover of fine horses and possessed several prize winners.

On August 14, 1914, he was married to Grace Elizabeth Meigs, of Madison, Wis., who, with one child, survives him.

Mr. Williamson was elected an Associate Member of the American Society of Civil Engineers on February 7, 1906, and a Member on October 31, 1911.

FREDERIC IRVING WINSLOW, M. Am. Soc. C. E.*

Died February 21, 1924.

Frederic Irving Winslow was born at New Bedford, Mass., on January 30, 1863, the son of George William and Jane Lucretia (Southwick) Winslow.

He received his early education in private and public schools with one year in High School. In 1877, he removed to Boston and attended the English High School from which he was graduated in 1881, having received the Lawrence Prize and the Franklin Medal. Subsequently, he was graduated from the four-year course in Mechanical Design in the South Boston School of Art.

Mr. Winslow's first position was in the City Surveyor's office at Boston, in 1881, and he continued in the employ of that city for about thirty-eight years. At first, he was engaged in surveys and, later, entered the City Engineer's office as Transitman in the Water Department and from 1885 to 1911 was Senior Assistant Engineer of the Boston Water-Works. From 1911 to 1916, he served as Engineer in charge of all the extension work, and then became Assistant Engineer on street construction for the City. From October, 1917, to August, 1918, he held the position of Supervising Engineer in charge of the office and the construction at the Squantum Destroyer Plant for the United States Navy Department.

From April, 1919, until his death, Mr. Winslow was Division Engineer of the Sudbury Department of the Metropolitan Water-Works, stationed at Framingham, Mass., and also carried on a consulting practice.

His specialties were hydraulics and water-works. Among his other accomplishments are the extension of water service to Long Island, Boston Harbor; the design and construction of Deer Island Reservoir; and the design and construction of two tunnels in compressed air.

Mr. Winslow was the author of numerous papers on engineering topics, which appeared chiefly in *Engineering News,* for which he was for a short time Boston correspondent, in the *Journal* of the New England Water Works Association, and in the *Harvard Engineering Journal;* and of a chronology of the Boston Water-Works (1912).

He was a member of the Boston Society of Civil Engineers from 1885, having served as Librarian from 1907 to 1913, of the New England Water Works Association from 1892 (he had been Treasurer from 1922 until his death), of the American Water Works Association, of the Boston Congregational Club, and of the Royal Arcanum. He was a Republican in his political affiliations.

Mr. Winslow was married on June 15, 1892, to Myrtle S. Smith, of South Newbury, Ohio, who survives him, with their son, Irving H. Winslow.

His death occurred at his home in Framingham on February 21, 1924, after an illness of only a few days.

He was a great student, making a practice to read about two hundred volumes annually for many years. He was a frequent attendant at engineering

* Memoir prepared by John C. Chase, M. Am. Soc. C. E., and Charles W. Banks, Assoc. M. Am. Soc. C. E.

meetings in Boston and had a wide acquaintance. He was a genial, companionable man, highly esteemed by all who knew him—one whose passing has left a noticeably vacant place, and whose memory will long be cherished by the many who knew him.

Mr. Winslow was elected a Member of the American Society of Civil Engineers on May 6, 1914, and was a Charter Member of the Northeastern Section of the Society. During the last year of his life he had rendered the Society valuable service in the preparation of memoirs of a number of New England members for whom none had ever been written.